DISTRICT COURT, CITY AND COUNTY OF DENVER, COLORADO Court Address: 1437 Bannock Street Denver, Colorado 80202	12/6/02
Plaintiff: SUNNYSIDE GOLD CORPORATION, **Defendant:** COLORADO WATER QUALITY CONTROL DIVISION OF THE COLORADO DEPARTMENT OF PUBLIC HEALTH AND ENVIRONMENT	▲ COURT USE ONLY ▲ Case Numbers: 94 CV 5459 Div.: Ctrm.: 7

ORDER GRANTING JOINT PETITION FOR FOURTH AMENDMENT TO CONSENT DECREE

THIS COURT, having reviewed the Joint Petition for Fourth Amendment to Consent Decree, and thereby being advised in the premises, GRANTS the Joint Petition and ORDERS the Consent Decree to be modified as follows:

1. Appendix A to the Consent Decree is modified to be in accordance with the Appendix A submitted with the Joint Petition;

2. Paragraph 9.c. of the Consent Decree is modified to be consistent with the agreement to transfer ownership of the water treatment facility to the Gold King Mines Corporation ("Gold King"). Once the water treatment facility is transferred to Gold King and CDPS Permit No. CO-027529 is terminated or transferred to Gold King by the Colorado Water Quality Control Division ("Division"), Sunnyside Gold Corporation ("SGC") obligation to continue operation of the water treatment facility to treat Cement Creek or any seepage from the American Tunnel (and the reclamation of the ponds and surface disturbances) will terminate under the Consent Decree and, accordingly, paragraph 14.f of the Consent Decree will be deleted at that time.

3. Paragraph 10 of the Consent Decree is modified so as to require only the monitoring contained in Appendix A and any applicable DMG and CDPS permits;

4. In addition, SGC will fund or implement the following additional remediation projects:

a. Provide a total of $500,000, which the parties anticipate will be more than adequate, for plugging the Mogul and the Koehler Mines by Gold King or another entity to be approved by the Division. The sealing of the Mogul and Koehler Mines would be in accordance with the workplans attached to the Joint Petition as Appendix B, and following execution of agreements with the owners of those mines and Gold King allowing and providing the terms for the plugging;

b. Provide $172,000 to Gold King for water quality improvement projects, including a liner at the Howardsville Cell No. 1 Mine Tailing, installation of a pipeline from the Gold King mine to the water treatment facility and water treatment at the American Tunnel treatment plant;

c. SGC will remove the power plant tailings; and

d. SGC will build a passive treatment wall at the southwest edge of Tailings Pond No. 4.

5. The Division shall notify the financial institution that has issued the letter of credit for financial surety referenced in paragraph 25 of the Consent Decree, that the letter of credit, $5,000,000 (Five Million Dollars) shall be released in full. The letter of credit funds shall be used for, but not be limited to, the funding of the projects referenced in paragraph 4 above;

DATED this 6th day of December 2002.

[signature]

District Court Judge

DISTRICT COURT, CITY AND COUNTY OF DENVER, STATE OF COLORADO

Case No. 94 CV 5459, Courtroom 7

JOINT PETITION FOR FOURTH AMENDMENT TO CONSENT DECREE

SUNNYSIDE GOLD CORPORATION,

Plaintiff,

v.

COLORADO WATER QUALITY CONTROL DIVISION OF THE COLORADO DEPARTMENT OF PUBLIC HEALTH AND ENVIRONMENT,

Defendant.

Sunnyside Gold Corporation ("SGC") and the Colorado Water Quality Control Division ("Division"), by their respective counsel, hereby jointly petition the Court for a fourth amendment of the Consent Decree entered in this matter on May 8, 1996, and in support of this joint petition, state as follows:

1. On May 8, 1996, this Court entered a Consent Decree resolving a declaratory judgment action between the parties. Paragraph 36 of the Consent Decree requires that the parties jointly petition the Court for any amendment to any portion of the Consent Decree or its appendices. Paragraph 37 of the Consent Decree provides for retained jurisdiction in this Court.

2. The mine tunnel plug in the American Tunnel, initially placed in 1996, has functioned and continues to function as designed while the mine pool has risen behind the plug to the point of physical equilibrium. Consequently, in accordance with the Consent Decree, SGC is installing the final plug in the American Tunnel leading to completion of that closure. Since the entry of this Consent Decree, SGC has completed all of the "A" list projects and they have been

approved by the Division, with the final plugging of the American Tunnel expected to be completed by December 10, 2002, and has also completed all of the reasonably beneficial "B" list projects.

3. Based upon further water quality data collection and analysis, SGC and the Division have agreed that it is appropriate to revise Appendix A to the Consent Decree, and to petition the Court to modify the Consent Decree and its terms to reflect the new Appendix A, attached hereto.

4. The parties hereto agree to modify paragraph 9.c. of the Consent Decree to be consistent with the agreement to transfer ownership of the water treatment facility to the Gold King Mines Corporation ("Gold King") as envisioned in a separate agreement, to be executed between SGC and Gold King. Gold King will maintain the treatment facility and ponds and be responsible for any surface disturbance reclamation required by the DMG Permit at the appropriate time. The water treatment facility will be transferred to Gold King following approval of this modification to the Consent Decree and the transfer of CDPS Permit No. CO-027529 to Gold King which will also terminate SGC's obligation to continue operation of the water treatment facility (and the reclamation of the ponds and surface disturbances.)

5. The parties agree that paragraph 10 of the Consent Decree, pertaining to monitoring requirements, is to be modified so as to require only monitoring contained in Appendix A and any applicable DMG and CDPS permits.

6. The parties hereby agree and stipulate that in order to further improve water quality in the Animas River Basin, SGC will fund or implement the following additional remediation projects:

a. Provide a total of $500,000, which the parties anticipate will be more than adequate, for plugging the Mogul and the Koehler Mines by Gold King or another entity to be approved by the Division;

b. Provide $172,000 to Gold King for water quality improvement projects, including a liner at the Howardsville Cell No. 1 Mine Tailing, installation of a pipeline from the Gold King mine to the water treatment facility and water treatment at the American Tunnel treatment plant;

c. SGC will remove the power plant tailings; and

d. SGC will build a passive treatment wall at the southwest edge of Tailings Pond No. 4.

7. The sealing of the Mogul and Koehler Mines would be in accordance with the workplans attached hereto as Appendix B, and following execution of agreements with the owners of those mines and Gold King allowing and providing the terms for the plugging.

8. The Division agrees that it will notify the financial institution that has issued the letter of credit for financial surety referenced in paragraph 25, that the letter of credit, $5,000,000 (Five Million Dollars) shall be released in full upon approval of this Fourth Amendment to the Consent Decree. The letter of credit funds shall be used for, but not be limited to, the funding of the projects referenced in paragraph 6 above.

9. The Division hereby stipulates and agrees that SGC and Gold King will apply to the Division for the transfer of CDPS permit No. CO-0027529 to Gold King and the Division will approve that transfer, unless Gold King applies for a new permit, in which event the permit No. CO-0027529 will be terminated. Following the transfer or termination of the subject permit,

SGC will have no further obligation to treat Cement Creek or any seepage that may issue from the vicinity of the plugged American Tunnel and accordingly, paragraph 14.f of the Consent Decree will be deleted at that time.

WHEREFORE, the parties jointly petition the Court to enter the attached Order granting this Fourth Amendment to the Consent Decree as described above.

Respectfully submitted this _____ day of December, 2002.

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WATER QUALITY CONTROL DIVISION OF THE COLORADO DEPARTMENT OF PUBLIC HEALTH AND ENVIRONMENT

John Chase, Acting Division Director
Water Quality Control Division
Colorado Department of Public Health and Environment

APPENDIX A
Reference Water Quality

A-1 Definition of "Reference Water Quality" and "Project Period"

"Reference water quality" is defined as the dissolved zinc (Zn) concentration and flow at reference point A-72, the US Geological Survey's stream flow gage no. 09359020, Animas River below Silverton, for the time period of September 5, 1991 to May 22, 1996. Measurements that were taken on the same day or on consecutive days were averaged to reduce the relative importance of those measurements when using the data in statistical analysis. This set of 17 flow/zinc pairs is termed the "baseline set". This set of data defined reference water quality against which later water quality will be compared. Both the raw data and the baseline data set are presented below in Table A-1.

"Project period" is defined as the time period that begins with the closure of the valve at the American Tunnel internal portal plug and the diversion of Cement Creek into the American Tunnel water treatment plant. The project period ends when there is final closure of the number two bulkhead in the American Tunnel.

"Post project condition" is defined as the condition of water quality that is expected after the A-list Projects have been completed; mine pool equilibrium has been reached; American Tunnel, Terry Tunnel, the Mogul Mine and the Koehler mine have been plugged; and diversion and treatment of Cement Creek have ceased.

A-2 Relationship between flow and dissolved Zinc in the baseline set.

The high-mountain headwaters portion of a river such as the Animas is subject to seasonal fluctuations in flow due to hydrologic response of the basin to climatic factors. In this basin, stream flow and concentrations of dissolved metals are inversely related. At low flows concentrations of metals are high, and at high flows concentrations are low, exhibiting a dilution effect from snowmelt. Figure A-1 presents the baseline Zinc concentrations and streamflow at A-72.

When comparing varying concentrations of dissolved metals in surface waters, standard practice is to adjust for the variation due to streamflow. This is an analytical technique used when pollutant concentrations vary as a function of streamflow, as shown in Figure A-1. If the data set consists of samples taken at a variety of streamflows, some or all of the concentration differences could be due to differences in streamflow and not to changes in upstream inputs. By computing a regression equation between concentration and streamflow, the effect of streamflow can be "subtracted out."

Table A-1: Dissolved Zinc and Streamflow data for A-72

Raw Data: A- 72				Baseline Data set: A-72			
Date	Agency	Flow (cfs)	Zinc (dis) (ug/L)	Date	Rise	Flow (cfs)	Zinc (dis) (ug/L)
9/5/1991	WQCD	131	380	9/5/1991	0	192	353
9/6/1991	WQCD	185	370				
9/7/1991	WQCD	261	310				
9/9/1991	WQCD	269	260	9/9/1991	0	266	265
9/10/1991	WQCD	263	270				
6/23/1992	WQCD	965	240				
6/24/1992	WQCD	955	290	6/24/1992	0	942	263
6/25/1992	WQCD	905	260				
10/14/1992	WQCD	80	480	10/14/1992	0	79	495
10/15/1992	WQCD	78	510				
7/20/1993	WQCD	434	290	7/20/1993	0	434	275
7/21/1993	WQCD	434	260				
11/10/1993	USGS	86	520	11/10/1993	0	86	520
5/16/1994		654	510	5/16/1994	1	629	505
5/18/1994	USGS	603	500				
6/2/1994	USGS	1370	300	6/2/1994	1	1370	300
7/26/1994	USGS	159	360	7/26/1994	0	159	360
1/18/1995	WQCD	72	680	1/18/1995	0	72	680
2/7/1995	SGC	90	600	2/7/1995	0	90	600
4/12/1995	USGS	127	790	4/12/1995	1	127	790
9/6/1995	BOR	239	360	9/6/1995	0	239	360
11/29/1995	BOR	nd	365	11/29/1995	0	76	428
11/29/1995	USGS	76	490				
1/16/1996	WQCD	65	540	1/16/1996	0	65	540
4/9/1996	BOR	167	950	4/9/1996	1	167	848
4/9/1996	USGS	167	830				
4/9/1996	WQCD	nd	780				
4/9/1996	USGS	167	830				
5/22/1996	USGS	1370	270	5/22/1996	1	1370	270
nd = no data							

In the Upper Animas Basin this dilution phenomenon is particularly distinct with an added "flush" effect on the rising limb of the hydrograph. One explanation of this flush effect is that as the snow begins to melt, additional metals are rinsed from the near surface soils adding to the concentration in the stream flow. This flush of metals occurs as the stream flow rises, generally starting in April and lasting through mid-June. Examination of the hydrograph (the daily flow records) is necessary to determine when the flows begin to increase in the spring and peak in June.

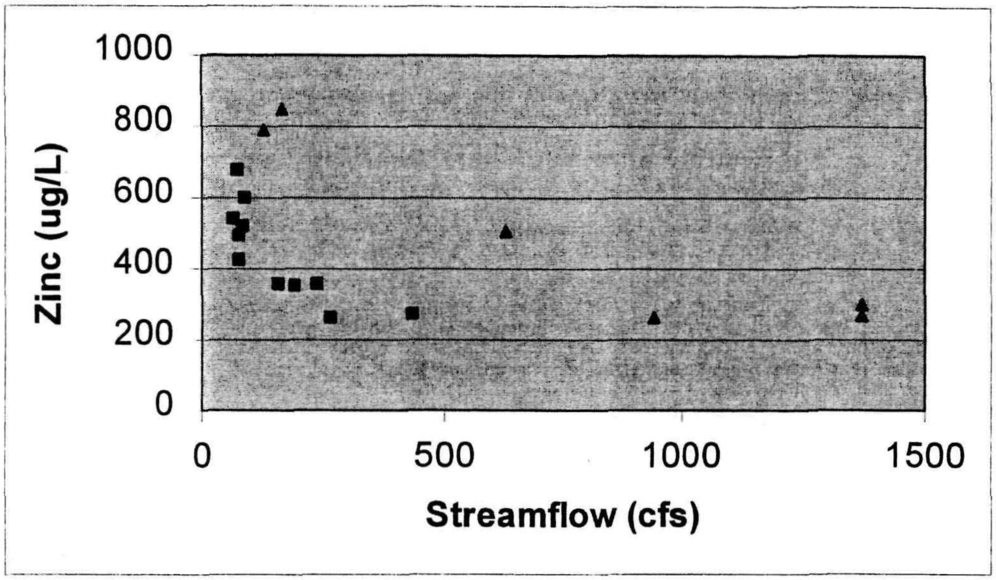


Figure A-1: Reference Water Quality Zinc Concentrations and Streamflow at A-72

To see this, in Figure A-1, samples on the rising limb of the annual hydrograph are plotted as triangles and other data as squares. The rising limb samples are generally higher in concentration than others for the same streamflow.

In order to use traditional statistical methods (which depend on normally distributed data), the data must be transformed. The natural logarithm is a commonly used transformation in water resources data analysis. Figure A-2 presents the baseline data after the natural logarithm transformation.

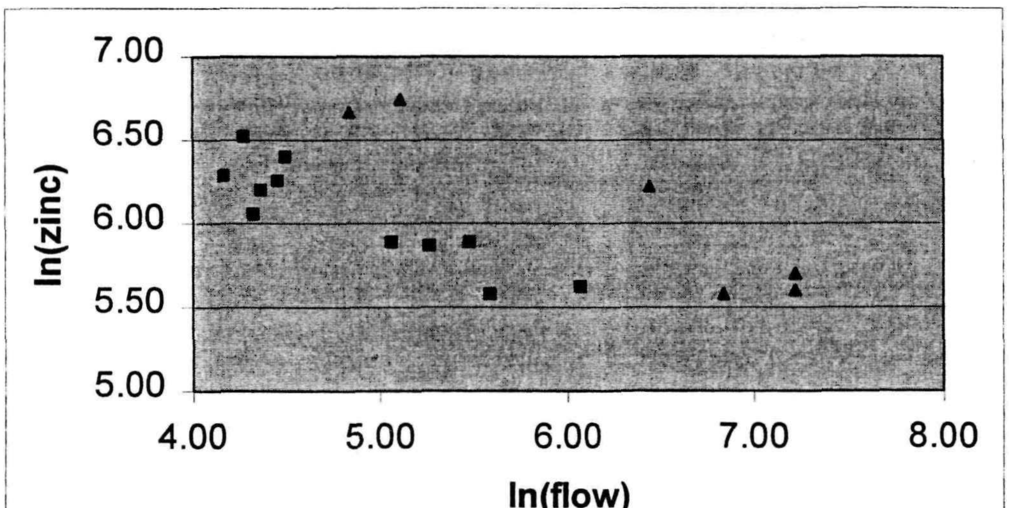


Figure A-2: Reference Water Quality at A-72, Transformed Zinc and Streamflow

Using regression analysis, the following equation was developed to describe the relationship between dissolved Zinc at A-72 as a function of stream flow and whether or not the samples were taken on the rising limb of the hydrograph (referred to as the "Rise").

$$\mathbf{\ln(Zn_{A\text{-}72}) = 8.30 - 0.462 * \ln(flow) + 0.687\ Rise} \qquad \mathbf{(1)}$$

$$\mathbf{Zn_{A\text{-}72} = e^{[\ln(Zn_{A\text{-}72})]}} \qquad \mathbf{(2)}$$

Where:

ln(Zn$_{A-72}$) = the natural logarithm of predicted dissolved zinc concentration (in ug/L) at A-72
ln(flow) = natural logarithm of stream flow (in cfs) at A-72
Rise = 1 if the flow is taken during the rising limb of the hydrograph and 0 if it is not
Zn$_{A-72}$ = the predicted dissolved zinc concentration (in ug/L) at A-72
e = base of the natural logarithm

Statistical t-tests are performed to determine whether the two independent variables (ln(flow) and rise) are important predictors of dissolved zinc. The results are shown in Table A-2. A slope coefficient for each of the variables in the equation is shown under "Coef" in the table. The value of the intercept, or constant in the equation is also shown. At the bottom of the table the adjusted correlation coefficient ("R-Sq(adj)") is presented. An R-Sq(adj) value of 84.4 % means that 84 percent of the variability in dissolved zinc concentration at A-72 is accounted for by the ln(flow) at A-72 and whether or not the sampled occurred during the rising limb of the hydrograph. Also at the bottom of the table is the standard error "S", in this case 0.1532. The standard error is a measure, which indicates the spread of the points around the fitted line of regression.

The column to the far right contains P values for each test. The P value is a measure of the believability of the hypothesis that no effect on Y (the ln(Zn)) is caused by that X (the ln(flow)). A small value for P, traditionally considered as smaller than 0.05 (5%), is evidence that the X variable does affect the values for Y. A P value smaller than 0.05 for Rise indicates that there are differences in zinc concentrations if the samples are taken in the rising limb of the hydrograph. The actual P value for the Rise is 0.000.

Table A-2. Regression Results for the Zinc Concentration at A-72

The regression equation is
$\ln(Zn_{A-72}) = 8.30 - 0.462 \ln(flow) + 0.687$ rise

Predictor	Coef	St Dev	T	P
Constant	8.2997	0.2435	34.08	0.000
ln(flow)	-0.46175	0.04910	-9.40	0.000
Rise	0.6872	0.1042	6.59	0.000

S = 0.1532 R-Sq = 86.4% R-Sq(adj) = 84.4%

To remove the effects of these two variables (the flow and the rise), the value predicted from the regression equation is subtracted from the measured value. This is called the residual, and is the distance the observed point lies above or below the regression line. The residual contains the variation due to all other variables than the two removed by the regression.

In order to be able to compare the residuals when the Y variable (zinc concentration) is transformed (ie, the analysis uses the natural logarithm of the zinc concentration), it is important to "standardize" the residuals by dividing all the residual values by the standard error[1]. In this way, the magnitude and importance of the residual can be judged. For instance, a 50 ug/L residual has much more importance if the predicted value is 100 ug/L (50% difference) than if the predicted value is 800 ug/L (6 % difference). The standardized residual is calculated as follows:

$$\mathbf{StR = ((ln[Zn_{A\text{-}72\ Act}]) - (ln[Zn_{A\text{-}72\ Pred}])) / S} \qquad \mathbf{(3)}$$

Where:

StR	= Standardized Residual
ln($Zn_{A\text{-}72\ Act}$)	= the natural logarithm of measured dissolved zinc concentration (in ug/L) at A-72
ln($Zn_{A\text{-}72\ Pred}$)	= the natural logarithm of predicted dissolved zinc concentration (in ug/L) at A-72, from equation (1)
S	= standard error of the prediction equation, in this case 0.1532

A plot of the standardized residuals displays the degree of departure of the actual zinc concentration from the predicted zinc concentration. Figure A-3 displays the standardized residuals for the baseline data set. Bars that descend below the centerline indicate samples where the actual value is less than the predicted value; bars that extend above the centerline indicate samples where the actual value is greater than the predicted value.

[1] This standardizing technique is an approximation, since the standard error in Table A-2 is the standard error at the mean. At flow values much higher or much lower than the mean, the standard error will be slightly different than the 0.1532 provided in Table A-2. For purposes of tracking water quality at A-72, the standard error of 0.1532 is adequate.

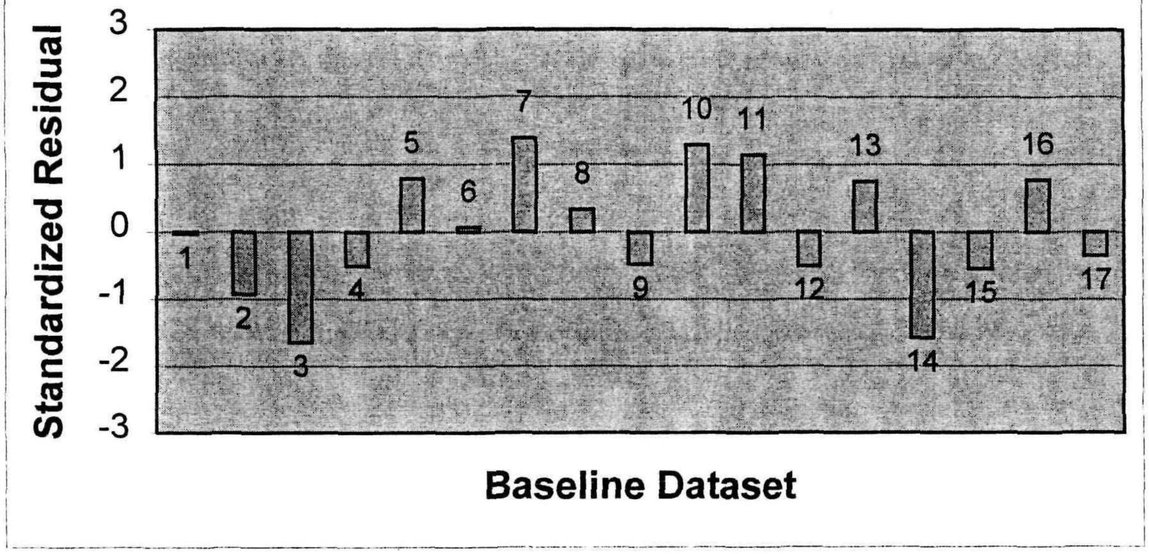


Figure A-3. Zinc Standardized Residuals (actual minus predicted) for Baseline Data Set

This approach of plotting standardized residuals showing the degree of departure of actual Zn concentration from predicted concentrations can be used not only for the baseline data set but also to track progress during the project period

A-3 Method for Evaluating Water Quality During the Project Period with Respect to the Reference Water Quality

(a) Each month during the project period, SGC will collect a water-quality sample at Station A-72. Analysis shall be conducted according to test procedures approved under 40 CFR Part 136, unless other test procedures have been approved by the Division. SGC will obtain the stream flow datum that applies to the relevant sampling event from the US Geological Survey data collection system. In the event that the stream gage at A-72 is frozen and/or inoperable, yet a water sample is obtained, flow may be estimated.

(b) SGC will report the flow value and the dissolved zinc value in a written submittal to WQCD. The submittal will be presented together with the next monthly Discharge Monitoring Report that is due to WQCD following receipt by SGC of the laboratory report for the dissolved zinc value. Records of monitoring information shall include:

- The date, exact place, and time of sampling or measurements;
- the individual(s) who performed the sampling or measurements;
- the dates the analyses were performed;
- the individual(s) who performed the analyses;
- the analytical techniques of methods used; and
- the results of such analyses.

(c) Within ten working days after the Court approves the Joint Amendment, which includes this revised Appendix A, SGC will calculate and report to WQCD an analysis of the data in the following form:

- For each month, SGC will calculate the predicted dissolved Zinc concentration, based on the flow at A-72 and the position on the hydrograph
- The standardized residual will be calculated using equation (3) above.
- The standardized residual will be plotted against the sampling period (which begins with the number 1 as the first sampling event of the project period) on a graph which has the standardized residual at the ordinate and the sampling period number as the abscissa. An example of such a graph is shown above as Figure A-3.
- SGC will compute a 12-month moving average of the standardized residuals, as follows. Beginning with the twelfth (12^{th}) month of the project period, SGC will compute the mean of the residuals for the 12-month period. For each subsequent month, the oldest residual will be dropped and the newest index number added to the set of 12 from which the mean residual is being computed.
- The moving average standardized residual value will also be plotted over time in a similar fashion to Figure A-3.

A-4. Method of Evaluating Water Quality with Respect to Reference Conditions for termination of the Agreement

The following describes the method of predicting the water quality at A-72 for the post project condition (once the A-List projects are complete; the American Tunnel portal, the Mogul Mine and the Koehler mine have been plugged; and diversion and treatment of Cement Creek have ceased).

(a) Calculation of the Expected Water Quality at CC01

Station CC01 is the sampling location on Cement Creek directly above the diversion into the American Tunnel water treatment plant. Water Quality at CC01 reflects the impacts of discharges from the Mogul Mine and other mining features as well as the contribution from the rest of the Upper Cement Creek Basin. In order to predict the zinc contribution of the Upper Cement Creek Basin in the post project condition, regression analysis was performed on the zinc and flow data at CC01 for the period before increased flows at the Mogul and Gold King Mines (August 1996 through July 1999).

The following equation was developed to describe the relationship between dissolved Zinc at CC01 as a function of stream flow and whether or not the samples were taken on the rising limb of the hydrograph. The "rise" term is used for the months June, July and August, since the elevation of CC01 is 10,600 ft compared to 9,200 ft at A-72 and the melt occurs later in the year.

$$\ln(Zn_{CC01}) = 1.38 - 0.159 \ln(flow) + 0.336\ Rise \qquad (4)$$

$$Zn_{CC01} = e^{[\ln(Zn_{CC01})]} \qquad (5)$$

Where:

ln(Zn_{CC01})	= the natural logarithm of predicted dissolved zinc concentration (in ug/L) at CC01
ln(flow)	= natural logarithm of stream flow (in cfs) at CC01
Rise	= 1 if the flow is taken during the months of June, July or August, and 0 if it is not
Zn_{CC01}	= the predicted dissolved zinc concentration (in ug/L) at CC01
e	= base of the natural logarithm

The results are shown in Table A-3. The "R-Sq(adj)" has a value of 68.1 %, which means that 68 percent of the variability in dissolved zinc concentration at CC-01 is accounted for by the ln(flow) at CC-01 and whether or not the sampling occurred during June, July or August.

Table A-3. Regression Results for the Zinc Concentration at CC-01

The regression equation is

$$\ln(Zn_{CC01}) = 1.380 - 0.159 \ln(\text{flow}) + 0.336 \text{ rise}$$

Predictor	Coef	St Dev	T	P
Constant	1.38171	0.03525	39.20	0.000
ln(flow)	-0.15891	0.04141	-3.84	0.001
Rise	-0.33616	0.08655	-3.88	0.00

S = 0.1779   R-Sq = 69.9%   R-Sq(adj) = 68.1%

(b) Calculation of Expected Water Quality at A-72

At the time of application for termination of Agreement, SGC will select the last 36 months of data (flow and dissolved zinc at A-72 and flow at CC01), and calculate the expected water quality at A-72 as follows:

- For each month, the expected CC01 load will be added to the actual load at A-72. The expected CC01 concentration will be based on the regression equation (4) and (5) above, the actual flow at CC01, and whether the data was collected during the "Rise." Concentration and flow will be used to calculate a CC01 load. This represents the zinc load from the Upper Cement Creek Basins that has been removed during the project period by diversion and treatment of Cement Creek in the American Tunnel Treatment Plant.

- For each month from July through and including March (A-72 low flow months), 54.6 pounds of zinc per day will be subtracted from the actual load at A-72. For each month from April through and including June (high flow months), 63.7 pounds of zinc per day will be subtracted from the actual load at A-72. This represents 70% of the load contributed by the Koehler Tunnel estimated by the Animas River Stakeholders Group (78 pounds per day during low flow months, and 91 pounds per day during high flow months)[2]. Subtraction of these pounds per day represents the predicted effect of plugging the Koehler Tunnel.
- For each month, 12.9 pounds per day will be added to the actual load at A-72. This represents 30 % of the median load of the American Tunnel (after placement of the 2nd portal plug). Final plugging is estimated to result in a 70% reduction in loading. This amount is added to the A-72 load since the entire load is now being removed by the American Tunnel Treatment Plant
- for each month, 36.0 pounds per day will be added to the actual load at A-72 . This represents 30% of the median load of the Mogul Mine (from August 2000 to present). Final plugging is estimated to result in a 70% reduction in loading. This is added to the A-72 load since the entire load is now being removed by the American Tunnel Treatment Plant

The steps below are the details of the calculation:

Actual Zinc Load at A-72:

$$\mathbf{actZn_{load} = Zn_{conc} * flow * 0.0054} \qquad \mathbf{(6)}$$

where:

$actZn_{load}$	= the actual zinc load at A-72 in pounds per day
Zn_{conc}	= the measured zinc concentration in ug/L
flow	= the measured flow in cfs
0.0054	= the conversion factor

Expected Zinc Load at A-72:

$$\mathbf{expZn_{load} = actZn_{load} + CC_{load} + AT_{load} - Kohl_{load} + Mog_{load}} \qquad \mathbf{(7)}$$

where:

$expZn_{load}$	= the expected zinc load at A-72 in pounds per day
$actZn_{load}$	= the actual zinc load at A-72 in pounds per day
CC_{load}	= the expected load from Cement Creek due to cessation of diversion and treatment of Cement Creek. (see below)
$Kohl_{load}$	= the expected load reduction from plugging the Koehler Tunnel (estimated at 66.3 lbs/day)
AT_{load}	= American Tunnel residual load (30% of load after 2nd plug)
Mog_{load}	= Mogul Mine residual (30% of load since Aug 2000)

[2] 70 percent was selected as the efficiency estimate for plugging the American Tunnel, the Mogul Mine and the Koehler Mine. Experience of the Hazardous Materials and Waste Management Division at the Bonanza Mining District has shown efficiencies of greater than 90 percent.

where:

$$CC_{load} = Zn_{CC01} * flow * 0.0054 \qquad (8)$$

and Zn_{CC01} is calculated using equations (4) and (5) above.

The expected zinc concentration at A72 is calculated by dividing the expected load by the actual flow at A72 and a unit conversion factor.

$$expZn_{conc} = expZn_{load} \div (flow * 0.0054) \qquad (9)$$

(c) Comparison of Expected Water Quality and Baseline Water Quality at A-72

Comparison of Expected Water Quality and Baseline Water Quality at A-72 will be done with a regression equation having three explanatory (X) variables: Streamflow, Rise, and 'After', where 'After' has a value of 1 if the data belong to the Expected data set and 0 if the data belong to the baseline data set. The response (Y) variable will be the natural logarithm of dissolved zinc in the baseline data set and the natural logarithm of the expected dissolved zinc concentration [equation (9)]. The regression equation will have the form of:

$$\ln(Zn_{A72}) = m + a * \ln(flow) + b * Rise + c * After \qquad (10)$$

Where:

ln(Zn_{A-72})	= the natural logarithm of the new predicted dissolved zinc concentration (in ug/L) at A-72
m	= the constant or intercept of the equation
a	= the coefficient for the natural logarithm of stream flow (in cfs) at A-72
ln(flow)	= natural logarithm of stream flow (in cfs) at A-72
b	= the coefficient for the "Rise" factor
Rise	= 1 if the flow is taken during the rising limb of the hydrograph and 0 if it is not
c	= the coefficient for the "After" factor
After	= 0 if the flow is from the baseline data set and 1 if it is from the last 36 months.

A t-test will be performed to determine whether this 0/1 Before/After difference is significant, over and above the effects of streamflow and rising limb, the other two variables in the equation. The t-test will be performed for each of the variables to determine whether or not the coefficient value is significantly different from zero. If so, that variable has an affect on the magnitude of the Y variable (the logarithm of zinc concentration). If not, (if the coefficient value is basically equal to zero) there is no effect of that variable on zinc. The test for whether differences occur "Before" versus "After" is found in the t-test for the variable 'After'. The T value and the P value will be reported. The P value is a measure of the believability of the hypothesis that no effect on Y is caused by that X. A value for P smaller than 0.05 (5%), will be evidence that the "Before/After" variable does affect the values for Y. A P value larger than 0.05 for After would indicate that there are no differences in zinc concentrations between the time periods.

SGC will complete a comparison of expected water quality and baseline water quality at A-72 and will submit the analysis (including the data, regression equation and T and P values for the coefficients) to the Division. This comparison will be used to evaluate the application for termination.

APPENDIX B
Detailed Work Plan and Benchmark Funding Schedule

#1 Send surface topography maps, tunnel long sections and any pertinent information to design engineer for guidance on probable plug location and plug size.

#2 Advance $15,000 per portal.
Open portal (portals if multiple levels) for 1 yd LHD access, establish ventilation and any other appropriate safety measures to secure portal and tunnel for selection of actual plug site and collecting rock samples. Build sediment traps as needed to control sludge that will be discharged.

#3 Close out #2 costs and advance $20,000.
Obtain Engineering design and submit to the Division of Minerals and Geology ("DMG") and the Water Quality Control Division ("Division") for approval.

#4 Establish coffer dam site, build coffer dam and divert water through piping.

#5 Close out #3 and #4 and advance $30,000.
Excavate plug area to solid rock, remove all loose rock and clean back, ribs and sill to remove mud, oxidation and other deposits to insure bonding of the concrete. Sand blasting works. Confirm size and taper assumptions used for design.

#6 Construct forms and place rebar. Arrange with DMG and the design engineer for pre-pour inspection. Determine grout pattern targets, mark hole collars to miss rebar and record drill angles and lengths to rock contact.

#7 Close out #5 and #6 and advance $10,000.
Place any alkaline material in the area between bulkhead and coffer dam planned for plug protection. Setup for pour and pour. Sample concrete for 7 day and 28 day tests during pour to confirm design strength has been met.

#8 Strip forms and drill holes for low pressure contact grouting. Grout holes until refusal.

#9 Close valve. Grout valve and close portal if permanent closure is selected by owner. Submit construction certification report to DMG and the Division. Close out #7, #8 and #9 and distribute Remaining Funds in accordance with the terms of this Agreement.

APPENDIX B
KOEHLER MINE WORK PLAN

<u>Remediation Plan</u>: The owner has agreed to have a plug installed in the Koehler Tunnel to stop drainage to Mineral Creek and Sunnyside Gold Corporation ("SGC") has agreed to facilitate this project with a specified level of funding.

<u>Remediating Party</u>:

Contractor:
Gold King Mines Corporation
Stephen C. Fearn, President
or other approved contractor

Owner:
Osiris Gold Inc
Sial Exploration Inc.
Frank Baumgartner

<u>Funding Party</u>:

Sunnyside Gold Corporation
P.O.Box 177
Silverton, CO 81433

Owner Contact:
Frank Baumgartner, President

Contact: Larry Perino
Reclamation Manager

Contractor Contact:
Stephen C. Fearn, President

1. <u>Description of Mining Activities</u>

<u>Physical Description of Conditions</u>

The Koehler portal discharges continuously although the flow varies seasonally. Water flowing from this portal carries dissolved metals. The regional geology is volcanic rocks with ring fault fractures and chimney type ore deposits containing base metals (Al, Cd, Fe, Pb, Cu and Zn), which this tunnel was driven to intersect. There are limited known mine workings associated with this tunnel but it is connected physically to surface at the San Antonio Shaft. Sampling of waters from the portal has identified it as a major contributor of dissolved metal loading to Mineral Creek.

<u>General Description of the Mining Site</u>

The history of the Koehler Tunnel is not known by SGC but it is believed to have been in operation in the 1950's. The Koehler portal was opened to capture and divert the mine drainage from the waste dump and test whether the discharge was amenable to passive treatment in 1996.

It was determined that the metal concentration was too high to economically operate and maintain a passive treatment system. The timber sets that were installed in 1996 will require inspection and movement or burial of the sill spreaders to allow access for construction. It is also known that a cave exists within the tunnel that may require passage through if an acceptable plug site does not exist out-bye of the cave.

Identification of Lands

The Koehler Tunnel is located at the headwaters of Mineral Creek just off of Highway 550 near the summit of Red Mountain Pass in San Juan County, Colorado. See attached location map.

Identification of the Waters of the United States Potentially Affected

The headwaters of Mineral Creek, Segment 8 of the Upper Animas River Basin.

2. Location Map

   Attached

3. Stormwater Management Controls

   Sediment catchments will be installed as needed. The majority of this project's activity will be underground.

4. Inspection and Record Keeping

   The Reclamation Manager or a member of the Technical Services Department from SGC will inspect this project on a regular basis until project completion. Quarterly reports with photographs will be submitted by the remediating party to both the Water Quality Control Division ("Division") and the Colorado Division of Minerals and Geology ("DMG"). Photographs of the property prior to remediation will be submitted with the first quarterly report.

Monitoring

Additional monitoring for this project is not contemplated. SGC and/or the Bureau of Reclamation ("BOR") monitors Mineral Creek at the USGS gaging station (M-34) above the confluence with the Animas River monthly. SGC will maintain this monitoring station until released from this requirement. BOR monitoring data will adequately characterize changes to Mineral Creek after SGC is released from this monitoring requirement.

Reporting

The design will be submitted in an acceptable form to the Division. The Division will approve the design prior to construction of the plug. This is not a long-term project.

Therefore, a final report will be submitted by the remediating party once all reclamation activities are complete as well as monthly progress reports while the project is active. Reports will be sent to the DMG as well as the Division.

5. Mine Remediation Plan

Legal Right to Enter and Conduct Activities

Negotiations are in progress to obtain an agreement for this project to be implemented.

Remedial Goals and Objectives

Reduction of metals loading to Mineral Creek by removing the artificial drain created by the adit and reducing the exposure of metal bearing rock to oxygen and any chemical reactions this exposure may precipitate. The project is to be completed at the earliest feasible time after agreement(s) finalization but no later than September 30, 2003.

Site Loading Estimate

The site loading estimate is based on the Animas River Stakeholder's Group's monitoring data contained in the Use Attainability Analysis for the Upper Animas Basin. The dissolved zinc load to Mineral Creek was estimated to be 78 pounds per day during low flow and 91 pounds per day for high flow.

Description of Project

The tunnel timber sets will be modified for access and the tunnel inspected for sites suitable for the placement of a plug. In order for a plug to be placed, a site meeting the following conditions will need to be found.

1) Location far enough underground to avoid the near surface fractures and joints caused by weathering.
2) Adequate rock compressive strength for structural stability.
3) A length of tunnel with minimal faulting or other geologic features.
4) Adequate ground cover over the potential site to resist the hydrostatic forces from the potential maximum head.

If an acceptable location can be found a plug will be designed and installed. After construction of the plug is complete, the plug will be contact grouted and if permanent closure is selected by the owner, any pipes through the plug for construction purposes will be grouted and the portal reclaimed.

Work Plan

1) Build catchments for potential sediment releases.
2) Open and evaluate tunnel for placement of a plug.
2) Design and install plug.
3) Grout seal-rock contact and piping if permanent closure is selected by the owner.

4) Reclaim the surface expression of the adit if permanent closure is selected by the owner.

Analysis

The plug proposed for the Koehler Tunnel will reduce the unsaturated zone by removing the drain. This will result in minimizing the oxygen available for reaction with the sulfide materials in the area. The hydrological conditions will be restored to an approximation of pre-mining conditions and should improve the water quality in the area.

Contingency Plans

Should the concept of plugs not be practical after engineering studies, SGC will consult with DMG for other possible solutions. If an acceptable, cost effective solution can be arrived at, such a system will be installed.

Monitoring

Additional monitoring for this project is not contemplated. SGC and/or the BOR monitors Mineral Creek at the USGS gaging station (M-34) above the confluence with the Animas River monthly. SGC will collect samples at this monitoring station until released from this requirement. BOR monitoring data will adequately characterize changes to Mineral Creek after SGC is released from this monitoring requirement.

Budget

SGC will fund this project up to the total project limits defined in the Joint Petition for Fourth Amendment to Consent Decree to be executed by SGC and the Division. The anticipated level of funding is $200,000, which is believed to be adequate for placement of the plug. SGC will also be funding up to $300,000 for plugging the Mogul No.1 Level Tunnel but the allocation can be adjusted as long as both projects are completed. The total funding level of $500,000 to complete the two projects is the maximum level of funding committed to by SGC.

Description of Land Use

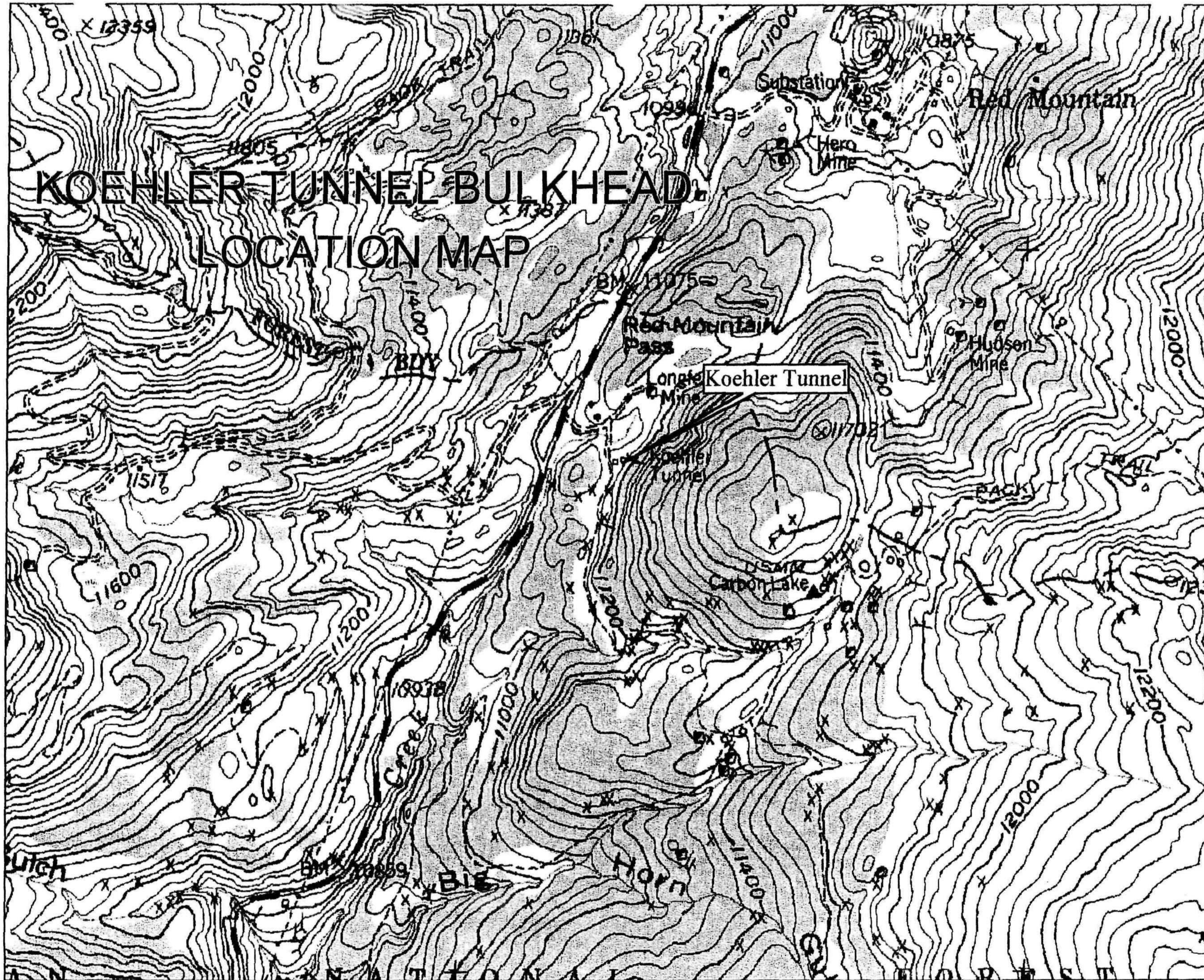
This remediation work plan is intended to use Best Management Practices on the site to reduce metal loading to Mineral Creek and to conform with land use policies for the area. Other than reclamation of the surface expression of the portal, no changes are anticipated that would modify existing suitable land uses.

Consistency with Other Plans

The plan is consistent with the Animas River Stakeholders Group's plan to implement Best Management Practice projects in the Upper Animas River Basin to improve water quality and meet Water Quality Standards set as a goal. This property is on the list of projects identified that will need to be implemented to reach those goals.

Attachments:

- Location Map
- Bulkhead Design for Acid Mine Drainage
- Detailed Work Plan and Benchmark Funding Schedule


KOEHLER TUNNEL BULKHEAD
LOCATION MAP
Koehler Tunnel
Red Mountain
Red Mountain Pass
Substation
Hero Mine
Hudson Mine
Longfellow Mine
Koehler Tunnel
Carbon Lake
BM 11075
12000
11400
11200
11000
12200
Big
Horn
Creek
PACK
TRAIL
BDY
N A T I O N A L F O R E S T

October 27-29, 1998

BULKHEAD DESIGN FOR ACID MINE DRAINAGE

John F. Abel, Jr.

in Proc Western U.S. Mining-Impacted Watersheds, Joint Conf on Remediation and Ecological Risk Assessment Technologies, Denver, CO

ABSTRACT

Impounding acid mine drainage behind a bulkhead in a mine tunnel has never been, and probably will never be, successful in reestablishing the pre-mining groundwater regime. However, even partially filling old mine workings should be beneficial. Partial filling should raise the mine depressed water table to the mine pool elevation. Partial filling of mine workings should decrease the quantity of groundwater entering mine workings, resulting in less mine drainage requiring treatment. Partial filling should deprive the submerged sulfide minerals of most of the oxygen necessary for producing acid, decreasing the rate of acid generation. Partial filling should improve the quality of acid water discharges from the mine. In effect, bulkheads can help but will never completely cure acid mine drainage.

Acid mine drainage bulkheads have several significant unknowns that potentially limit their usefulness:

1) What is the acceptable leakage around a tunnel bulkhead?

2) What are the natural flow paths for impounded acid mine water that may bypass a bulkhead into the open tunnel downstream from the bulkhead or to the ground surface?

3) How long will the bulkhead last?

4) Will unknown geologic conditions and(or) mine connections prevent the mine pool from reaching the planned elevation?

Concrete tunnel bulkheads designed to contain acid mine drainage water must be:

(1) long enough to prevent leakage along the contact between the concrete and the rock,

(2) thick enough to prevent shear failure in either the concrete or rock,

(3) prevent tensile failure of the downstream bulkhead face,

(4) deep enough to prevent hydrofracturing of the formation and

(5) acid resistant enough to last the requisite time.

The available design data includes, possibly in descending order of confidence, the strength and corrosion resistance of the concrete and steel, the strength of the rock, the maximum possible water head, the magnitude of the maximum credible earthquake and the in situ stress field.

INTRODUCTION

Ideally, impounding acid mine drainage behind a drainage tunnel bulkhead should reestablish the pre-mining groundwater regime. That hasn't happened and isn't likely to happen in the future. Even partially filling old mine workings should, however, be beneficial. Partial filling should raise the mine depressed water table to the mine pool elevation. Partial filling of mine workings should slow the rate and decrease the quantity of groundwater entering mine workings. Partial filling should deprive the submerged sulfide minerals of most of the oxygen necessary for producing acid, decreasing the rate of acid generation. In addition, partial filling should improve the quality of acid water discharges from the mine. In effect, bulkheads can help but will never be a complete cure for acid mine drainage.

Historically, and logically, mineral deposits have been exploited from the top down. This has resulted in many near surface access openings at the deposit outcrop. Some of the surface openings interconnect and some don't. Plugging the lowest draining portal may or may not significantly raise the level of the mine pool. Later in the life of a mine and a mining district, the deeper mine workings must be dewatered by pumping in order to continue mining. In such deeper mining operations, low level drainage tunnels may have been driven. Drainage tunnels have the potential, if plugged, for impounding water in a large part of the total mine excavation. Under no reasonable scenario, however, will plugging a single mine opening raise the mining depressed water table to its pre-mining level.

Acid mine drainage bulkheads have several significant unknowns that potentially limit their usefulness:

1) What is the acceptable leakage around a tunnel bulkhead, along the contact between the bulkhead and the rock and through the lower permeability rock immediately adjacent to the tunnel?

2) What are the natural flow paths for impounded acid mine water that may bypass a bulkhead into the tunnel downstream from the bulkhead or to the ground surface?

3) How long will the bulkhead last?

4) Will unknown geologic conditions and(or) mine connections prevent the mine pool from reaching the planned elevation?

Regardless of the location of a single bulkhead, water impounded upstream of the bulkhead may see the open downstream portion of the tunnel as a significant low resistance path for mine water discharge. The quantity of water forced back into the mine workings or to discharge at the ground surface by a bulkhead versus the quantity discharging into the downstream tunnel of a single bulkhead will depend on the rock substance, directional fracture and structure controlled permeability of the rock formation.

BULKHEAD DESIGN CONSIDERATIONS

Concrete tunnel bulkheads designed to contain acid mine drainage water must be:

(1) long enough to prevent leakage along the contact between the concrete and the rock,

(2) thick enough to prevent shear failure in either the concrete or rock,

(3) either thick enough to prevent tensile failure of the downstream face or contain sufficient tensile reinforcement to support the tensile stress,

(4) deep enough to prevent hydrofracturing of the formation,

(5) acid resistant enough to last the requisite time interval and

(6) strong enough to resist the maximum credible earthquake.

The available design data includes in descending order of confidence, the strength of the concrete and steel, if used, the strength of the maximum credible earthquake, the strength of the rock, the maximum possible water head and the in situ stress field.

Design of a concrete bulkhead can proceed once the mine layout and maximum possible hydraulic head are known and the bulkhead location selected on the basis of known hydrologic conditions and rock properties. The bulkhead location must first be prepared by removing rock loosened during the tunnel excavation.

Hydraulic Pressure Gradient

The pressure gradient (P_g) across a bulkhead is the hydraulic pressure, in psi, divided by the thickness of the bulkhead, in feet. Figure 1 presents the types of water-impoundment bulkheads generally used. It should be noted that the typical "taper plug", such as shown on Figure 1 is 7°. A bulkhead for a tunnel must be in intimate contact with the tunnel walls to prevent leakage along the concrete-rock interface around the plug. Bulkhead failure by leakage around the bulkhead, in the case of mine bulkheads, is more likely than failure of the bulkhead under thrust. Loofbourow in the Society of Mining Engineers (SME) Mining Engineering Handbook (1973, Sec 26.7.4) states "no indication of structural failure resulting from thrust was noted" in the case of ten bulkheads subjected by hydraulic pressures in excess of 1000 psi and which relied solely on normal rock surface irregularities, referred to as a "parallel plug" on Figure 1. High hydraulic pressure differentials across a bulkhead can be achieved by placing a long plug with a low resistance to water flow along the concrete-rock interface or by placing a short plug with high resistance to water flow along the concrete-rock interface achieved by grouting the concrete-rock contact. The Mining Engineering Handbook also recommends, in the same section, 40 to 25 feet of plug length for each 1000 psi of hydraulic head, i.e. pressure gradients from 25 to 40 psi/ft. The recommended concrete-rock grout pressure is "a few hundred psi". In practice, the grouting pressure must be kept below the formation breakdown pressure to prevent hydrofracturing. This limitation is particularly important for near surface bulkheads in order to prevent opening of fractures and possible release of impounded water through the formation to the open tunnel downstream or possibly even to the ground surface.

Garrett and Campbell Pitt (1961) reported the results from 26 mine bulkheads, 12 "parallel plugs", that relied solely on the irregularity of the tunnel walls, and 14 "taper plugs". However, they presented field data for 7 ungrouted bulkheads indicating acceptable leakage and pressure gradients from 18.0 to 26.2 psi/ft, averaging 21.4 psi/ft. The pressure gradient for the original ungrouted 6-ft thick bulkhead in the Friday Louden Tunnel was 15.3 psi/ft and did not leak when subjected to the measured 212 ft of head. Chekan (1985) analyzed Garrett and Campbell Pitt's pressure gradient data and produced a graphical version of their data. Figure 2 presents a modified version of the data that indicates that an ungrouted plug should be able to withstand a pressure gradient of approximately 21.3 psi/ft at a factor of safety of one. They also recommended a minimum factor of safety of 4 in good rock, yielding a recommended maximum pressure gradient of just over 5 psi/ft for average field conditions. Garrett and Campbell Pitt (1961) reported unacceptable leakage along the concrete-rock contact at 9.8 psi/ft when their ungrouted experimental bulkhead was pressurized to 75 psi. Obviously, the effectiveness of bulkhead concrete filling can vary widely, at least with respect to construction practice. It would not be realistic to attempt to build an ungrouted acid mine drainage bulkhead.

Garrett and Campbell Pitt indicated that pressure grouting of the concrete-rock contact of their experimental bulkhead would permit pressure gradients of 163 psi/ft without obvious leakage. Applying a factor of safety of four produces a design pressure gradient of over 40 psi/ft when the concrete-rock contact was grouted. The indicated benefit from pressure grouting the concrete-rock interface is an eight fold decrease in bulkhead length required to prevent unacceptable leakage.

What constitutes "unacceptable" leakage is a function of the bulkhead. The South African mining experience, reported by Garrett and Campbell Pitt (1961), indicates acceptable long term leakage along the concrete-rock contact and through the rock immediately around the bulkhead ranges from 3 gpm to 13 gpm and that 17 gpm was acceptable for short term leakage. Coogan and Kintzer (1987) indicate that 33 gpm leakage was not acceptable for a hydro tunnel but was acceptable when reduced to 11 gpm.

The leakage requirement for acid mine drainage bulkheads is generally more restrictive. In every case the goal is to reduce the flow to occasional drips at the bulkhead face. One contract specification is to limit the quantity of inflow at or within a specified distance from the downstream bulkhead face. The 1250 Bulkhead in the Reynolds Adit had such a requirement. The Reynolds Adit is in a weak, fractured and faulted rock formation. Before construction several tunnel sections were dripping measurable acidic groundwater. Limited formation grouting around the tunnel at the bulkhead location was employed before bulkhead construction. The purpose of the limited 6-foot radial formation grouting was to lower the permeability of the blast damage zone immediately adjacent to the tunnel walls. Obert and Duvall (1953) reported rock damage 48 hole radii from spherical explosive charges. Petykoph et al (1961) reported rock damage from 66 to 72 radii from cylindrical explosive charges. Since tunnel blasting always involves cylindrical charges the thickness of the blast damage zone was estimated as approximately 4.5 feet, in this case. Formation grouting beyond the potential blast damage zone was not undertaken because the specific fracture flow channels were not known. After water impoundment, groundwater inflow increased at a faulted tunnel section about 100 feet downstream from the bulkhead.

Garrett and Campbell Pitt indicated that high-pressure grouting of the rock adjacent to a bulkhead will result in a considerable increase in the allowable pressure gradient across the plug. However, high-pressure grouting is not an option for near surface plugging of old mine tunnels. Near surface high-pressure grouting could result in hydrofracturing of the rock around the tunnel.

The length (L) of a low-pressure grouted bulkhead necessary to meet the 40 psi/ft hydraulic pressure gradient criteria necessitates the calculation of the maximum pressure head (ρ), as follows:

$$\rho = \frac{H\gamma_w}{144} \tag{1}$$

H - design water head
γ_w - water density

The required bulkhead length with low pressure grouting is:

$$L = \frac{\rho}{40} \text{ ft} \tag{2}$$

Perimeter Shear Strength Design

Bulkhead design to resist shear stresses resulting from water impoundment involves evaluating concrete and rock shear strength along the perimeter of the tunnel and shear in the concrete at the critical section, as defined by the American Concrete Institute (ACI 318-95, Sections 11.8.1 and 11.8.5). Critical section shear includes the reinforcing bars, if present, in the designated section and, therefore, cannot be evaluated until the bulkhead reinforcing steel is tentatively selected.

The first requirement for evaluating bulkhead shear strength at the perimeter of the tunnel involves testing the rock to see whether the rock is stronger than the design concrete shear strength. Typically, the shear strength, cohesion, of the rock exceeds the design shear strength of the concrete. The measured compressive strength of the intact latite porphyry that is adjacent to the Ransom Tunnel bulkhead design example in Appendix A ranges from 10,260 psi to 35,570 psi and the estimated shear strength, cohesion, from approximately 2,500 psi to 8,900 psi. The concrete design shear strength (f_s'), for the 3,000 psi concrete compressive strength (f_c') is 110 psi, specified by the American Concrete Institute as follows:

$$f_s' = 2\sqrt{f_c'} = 2\sqrt{3000} = 110 \text{ psi} \quad \text{(ACI 318-95, Sec 11.3.1.1)} \quad (3)$$

Obviously, the concrete is the critical design component for perimeter bulkhead shear at the Ransom Tunnel. This not always the case as was the case for the best ground in the Chandler Tunnel at the Summitville Mine, as shown on Figure 3.

When concrete design shear strength (f_s') is less than the rock cohesion (c_r), the bulkhead length (L) needed to support the maximum perimeter shear stress from the application of the maximum pressure head (ρ), for a rectangular tunnel cross section with a height of (h) and width of (ℓ) is:

$$L = \frac{\rho h \ell}{2(h+\ell) f_s'} \text{ ft} \quad (4)$$

When rock cohesion (c_r) is less than the concrete design shear strength (f_s'), rock cohesion replaces concrete design shear strength in Equation 4.

Plain Concrete Deep Beam Bending Stress Design

The American Concrete Institute's "Building Code Requirements for Reinforced Concrete (ACI 318-95)" are recommended for design because the bulkheads are analogous to reinforced deep-beam concrete structures and because of the inherent conservatism of the code. It is difficult to obtain good adhesion between a concrete bulkhead and the roof and floor of a tunnel. The difficulty lies in completely cleaning of the floor and keeping it clear of mud and rock until the concrete is poured and in completely filling all the voids in the roof, even with low-pressure grouting. The deep-beam bulkhead should be conservatively assumed to act only one-way, between the walls (ribsides) of the tunnel. However, two-way reinforcing steel should be provided in bulkhead design to transfer some load to the tunnel roof and floor despite the difficulty in achieving intimate contact with the roof and removing all the loose rock from the floor. The one-way design assumption in effect produces a potential factor of safety of two, provided the more difficult roof and floor contacts between the bulkhead concrete and the rock are actually achieved by the recommended low-pressure contact grouting.

The recommended deep-beam bending analysis is based on a uniformly-loaded beam supported by the tunnel walls. This conservative design approach can be further justified by the inability of obtaining access to the upstream side of a bulkhead and the long life expected of the plugs.

The length of an unreinforced, plain, concrete bulkhead necessary must keep the tensile bending stresses in the downstream face below ACI allowable concrete tensile stress (f_t). ACI (318-95, Section 9.3.5 and 318.1-89, 1989, Section 6.2.2) directs that a strength reduction factor of 0.65 be used in design. ACI (1989, sec 6.2.1 and 318-95, Section 22.5.1) directs that the design tensile concrete bending stress not exceed:

$$f_t = 5\sqrt{f_c'} \tag{5}$$

f_c' = concrete design compressive strength

This amounts to 274 psi for 3,000 psi concrete. ACI (318-95, Section 9.2) also requires a 1.4 load factor for definable fluid loads.

The required length of an unreinforced plain concrete bulkhead to prevent tensile cracking on the downstream bulkhead face for a one-way (rib to rib) deep beam follows. The first step is to calculate the maximum nominal bending moment (M_n) on the one-way beam, as follows:

Fluid load per lb/ft^2

$$w = 1.4(\rho)144 \tag{6}$$

Maximum nominal bending moment

$$M_n = \frac{w\ell^2}{8} \text{ ft·lb} \tag{7}$$

Nominal bending moment adjusted for capacity reduction factor (ϕ) of 0.65 to obtain the factored design bending moment (M_u :

$$M_u = \frac{M_n}{\phi} = \frac{M_n}{0.65} \text{ ft·lb} \tag{8}$$

Maximum flexural stress

$$\sigma = \frac{M_u}{S} \text{ psi} \tag{9}$$

S = section modulus (in^3)

$$\text{Section modulus (in}^3\text{)} = \frac{I}{c} \tag{10}$$

I = moment of inertia (in^4)
c = centroidal distance (in)

Moment of inertia (in^4) = $\frac{bL^3}{12}$ (11)

b = beam width (in)
L = beam depth (bulkhead length) (in)

Centroidal distance (in) = $\frac{L}{2}$ (12)

Therefore, allowable flexural stress (f_t) in psi is

$$f_{cl} = \frac{M_u}{S} = \frac{M_u}{\frac{I}{c}} = \frac{M_u}{\left(\frac{\frac{bL^3}{12}}{\frac{L}{2}}\right)} = \frac{6M_u}{bL^2}$$

$$L^2 = \frac{6M_u}{b(f_t)} \quad (13)$$

Required length (L) of plain concrete bulkhead, obtained by solving equation (13) for the beam depth (L), the bulkhead length, is presented for the Ransom Tunnel Bulkhead in Appendix A. The length of the bulkhead can be decreased by the use of tensile reinforcement, provided the plain concrete bulkhead is longer than needed to limit the hydraulic pressure gradient to an acceptable level. Normally a trial bulkhead length is selected and the reinforcement required to support the tensile bending stress calculated.

Reinforced Concrete Deep Beam Bending Stress Design

The length of an alternative reinforced plain concrete bulkhead depends on providing sufficient reinforcing steel to support the entire tensile bending stress developed in the deep beam bulkhead. The ACI capacity reduction factor for bending of a reinforced concrete deep beam (ACI 318-95, Section 9.3.2.1) is 0.90. The method employs the rectangular compressive stress distribution approximation. The ACI method is described in Section 10.2, for a reinforced simple concrete beam and Section 10.7 for deep beams. These ACI Sections define a simple, simply supported, deep beam as one whose depth exceeds 4/5 the span. ACI defines a reinforced continuous concrete beam as a deep beam if the depth exceeds 2/5 the span. A bulkhead that can rotate at its supports, a simply supported beam, would be unlikely to be able to retain a fluid. The recommended low-pressure grouting is designed to fix the roof, walls and floor of the bulkhead, preventing rotation. In the case of the 10-foot width of the Ransom Tunnel Bulkhead design in Appendix A, the 8-foot thick bulkhead is a deep beam for design in either case.

The compressive load toward the upstream (water side) face of the bulkhead must be balanced by the tensile reinforcement from the rebar cage a few inches from the downstream (air side) face. Deep beam design assumes that a uniform compressive stress equal to 0.85 times the specified concrete compressive strength acts over an area 1 ft wide by 0.85 times the centroidal distance in depth (a) below the loaded surface. The constant, 0.85, is reduced 0.05 for each 1,000 psi the concrete strength exceeds 4,000 psi. The method, as further described by Wang and Salmon (1985, p 43-44), assumes the tensile reinforcing steel yields before the concrete crushes under bending induced compressive stress. Tensile reinforcement design for the typical reinforced concrete deep beam bulkhead follows:

Compressive force $C = \phi(f_c')ba = 0.85(f_c')ba$ (14)

Tensile force $T = A_s f_y$ (15)

b = beam width (in.)
a = compression zone depth (in.)
A_s = steel area (sq in./ft)
f_y = steel yield stress (psi)
f_c' = concrete strength (psi)

The method presented by Wang and Salmon (1985) assumes that the compressively stressed concrete area is no deeper into the beam than necessary to carry the bending moment developed compressive force at the ACI specified compressive stress of 0.85 times the specified compressive strength. The calculations equating C to T using equations (14) and (15), using 3000 psi concrete and 60000 psi yield strength rebar follow:

$$C = T \qquad 0.85(f_c')ba = A_s f_y \qquad 0.85(3000)12a = 60000A_s$$

$$a = \frac{A_s f_y}{0.85(f_c')12} = \frac{60000A_s}{0.85(3000)12} = 1.96078A_s \qquad (16)$$

Summation of moments about center of the compressively stressed area, substituting the compression zone depth from equation (16)

$$M_u = A_s f_y\left[d-\frac{a}{2}\right] = A_s f_y\left[(L-m_c)-\frac{a}{2}\right] \qquad (17)$$

d = depth, top of beam to center of reinforcing steel (in) = $L-m_c$
M_u = factored design beam bending moment (in·lb)
m_c = minimum cover, form face to rebar surface (inches)

$$M_u = A_s f_y\left[(L-m_c)-\frac{a}{2}\right] = 60000A_s\left[(L-m_c)(12) - 1.96078\frac{A_s}{2}\right] \text{ in}\cdot\text{lb}$$

$$M_u = 60000A_s[(L-m_c)12-0.98039A_s] \text{ in}\cdot\text{lb} \qquad (18)$$

The design flexure resisting bending moment is:

$$M_u = \frac{M_n}{\phi} = 60000A_s[(L-m_c)12-0.98039A_s] \text{ in}\cdot\text{lb} \qquad (19)$$

The solution for reinforcing steel area per foot of the one-way beam from rib to rib requires equating Equation 19 to the bending moment resulting from factored load. The nominal maximum bending moment is $M_n = \frac{w\ell^2}{8}$ ft·lb, as calculated by Equation 7. However, the strength factor (ϕ) for the steel tensile reinforcement is 0.90 and the factored bending moment from loading becomes:

$$M_u = \frac{M_n}{\phi} = \frac{\frac{\omega\ell^2}{8}}{0.90} = \frac{\omega\ell^2}{0.9(8)} \text{ ft}\cdot\text{lb} \qquad (20)$$

Therefore, the minimum required steel area per foot (A_S) is calculated from equating Equation 19 and Equation 20.

The design bulkhead thickness typically required to prevent leakage due to the hydraulic pressure gradient and to resist the perimeter shear forces makes the use of a simple beam design for bending of a possibly fixed-end beam extremely conservative. The bending deformations causing appreciable reinforcing steel strain, and therefore tensile stress, will not be linear due to the bulkhead thickness and the lateral restraint provided by the tunnel ribs. Bulkhead failure would most likely occur by concrete yielding of a pressure arch that would develop in the upstream side, rather than as the result of yielding of the reinforcing steel. Reinforcing steel is required at both the downstream and upstream bulkhead faces to control temperature and shrinkage induced stresses in the large bulkhead pour.

Critical Section Shear

ACI requires evaluation of critical section shear if the ratio of the bulkhead span (ℓ) divided by the distance (d) from the upstream bulkhead face to the centroid of the reinforcing steel $\left[\frac{\ell}{d}\right]$ is less than 5 (ACI 318-95, Section 11.8.1). This appears to always be the case for bulkheads that meet the pressure gradient requirement. This evaluation is very complex and has not been critical to bulkhead design. An example of the critical section shear evaluation method is presented in Appendix A.

Bulkhead Depth Based on Hydraulic Pressure

Hydrofracturing, generally referred to as hydrofracing, of sedimentary formations from drillholes is frequently undertaken for the purpose of stimulating oil well production. Formation breakdown pressure (B_p) is a function of (1) the tensile strength of the rock immediately adjacent to the drillhole, (2) the in situ stress field in the plane perpendicular to the drillhole and (3) the pore pressure present in the formation. Bredehoeft, et al (1973) presented a study of drillhole hydrofracturing of a competent rock. They presented the following well known equation for breakdown pressure:

$$B_p = T_s + 3S_{min} - S_{max} - P_f \tag{21}$$

All terms in psi

T_s = tensile strength
S_{min} = minimum stress normal to the borehole
S_{max} = maximum stress normal to the borehole
P_f = formation pore pressure

The equation can be simplified for the case of hydraulic pressure behind an acid mine drainage bulkhead in a tunnel. First, the tensile strength can be assumed to be zero because the rock adjacent to a tunnel is jointed and generally damaged by blasting. The packed-off section of a drillhole, on the other hand, can be entirely within one joint block and is not subject to blasting damage. Second, the pore pressure present near surface and adjacent to a tunnel must be low and can also be assumed to be zero. Finally, in the absence of in situ stress measurements it is necessary to estimate the stresses in the plane normal to the tunnel. The simplest assumption is for hydrostatic stress conditions equal to the overburden stress. The assumption is generally conservative since the overburden stress must be present and the more general stress state measured is for near surface horizontal stresses to equal or exceed the overburden stress. Normal formation breakdown pressures encountered in shallow oil field work range from 1.4 to 2.8 times the overburden stress. This indicates that the hydrostatic stress assumption, where the formation breakdown (hydrofracing) pressure equals two times the overburden stress, is not unreasonable.

The resulting simplified breakdown pressure equation is:

$$B_p = 2S_{ovb} \tag{22}$$

S_{ovb} = overburden stress (psi)

Acid mine drainage bulkheads must be placed at a depth which will not result in hydrofracing the rock adjacent to the tunnel, i.e. opening of the joints and fractures and injection of acid mine water into the rock mass around the plug and possibly to the ground surface.

The hydraulic breakdown pressure (B_p) available to hydrofrac the rock immediately upstream from the plug and adjacent to the tunnel is the maximum potential head. Therefore, the overburden stress must be sufficient to prevent hydrofracing. The required overburden stress (S_{ovb}) is:

$$S_{ovb} = \frac{B_p}{2} \tag{23}$$

The overburden pressure is the product of the depth (H) and density (γ) of the overlying rock. Since the density can be readily measured, the depth of the bulkhead must be selected to limit the possibility of hydrofracing, as follows:

$$S_{ovb} = \frac{\gamma H}{144} = \frac{B_p}{2} \tag{24}$$

γ = density (PCF)
H = depth (ft)

$$H = \frac{72 B_p}{\gamma} \tag{25}$$

Corrosion Resistant Design

The useful life of a concrete bulkhead is controlled by the corrosive nature of the acid mine drainage being impounded, the formulation of the concrete mix and on the corrosion resistance of the piping penetrating through the bulkhead. The corrosion characteristics of the impounded acid mine drainage can not be controlled. It is likely that the quality of the drainage water will change during the course of mine filling and after the maximum head has been reached. Sampling of the water impounded immediately behind the American Tunnel bulkhead has shown wide pH fluctuations since the valve was closed. Initially the pH rose well above 8, apparently as the result of the 20 tons of lime and 20 tons of limestone placed upstream from the bulkhead. The pH has dropped to 2.8 in the last year possibly as the result of solutioning of precipitates that have accumulated on the walls of underground openings that have since been inundated. However, no iron has been detected in the water samples taken at the American Tunnel Bulkhead. Iron present as Fe^{+3} ions tend to surface coat limestone limiting its further dissolution (USBM, 1994).

Concrete Mix Considerations

Chemical attack on the bulkhead concrete exposed to sulfate concentrations in the impounded mine water in contact with a bulkhead can be resisted by using Type V, sulfate resistant cement, as required by the ACI (ACI 318-95, Section 4.3). Table 1 presents the ACI requirements for concrete exposed to various sulfate concentrations. Brown (1992b, p 1) indicated that the 1250 Bulkhead in the Reynolds Adit at the Summitville Mine would be subjected to a 4643 mg/l (ppm) sulfate ion concentration. In addition, pozzolan (fly ash) can be added to the concrete mix to decrease concrete permeability and improve sulfate resistance, as recommended by ACI (ACI 318-89, Table 4.3.1) and Troxell et al (1968, p 104) for concrete in contact with "Very Severe", greater than 10000 ppm, sulfate concentrations.

A typical 3000 psi bulkhead concrete mix is 1 sack of Type V cement (94 lbs), to 235 lbs of fine aggregate (sand) to 330 lbs of well graded coarse aggregate and 15 lbs of fly ash (pozzolan). The mix proportions are 1:2.5:3.5 (cement, sand, gravel). One yard of concrete would contain 5.7 bags of cement (536 lbs), 1340 lbs of sand, 1881 lbs of well graded 1/2-inch maximum coarse aggregate and 86 lbs of fly ash, pozzolan. One yard of the specified concrete would have a dry weight of 3843 lbs/yard and a mixed weight of 4085 lbs/yard when the required 29 gallons of water is added. See ACI 211.4R-93 for additional details. The approximate in-place density of the concrete will be 151 lb/cu ft.

The typical mix would normally be considered "oversanded". However, the higher than normal sand content is designed to increase pumpability, i.e. slump, at the low water/cement ratio of 0.45 required to resist "Severe" or "Very Severe" concentrations of sulfate in acid mine water. High slump concrete can be pumped as a wet mix through a slick-line or pneumatically blown as a dry mix with the water added as placed in the bulkhead as shotcrete. Pneumatic transport is possible over greater distances but with a more variable field controlled water/cement ratio.

The Standard Handbook for Civil Engineers (Merritt, 1983, Table 8-4) indicates that a well-graded aggregate with a maximum size of 2 inches can be used with the mix proportions specified. However, it is recommended that 1/2-inch maximum aggregate size be used to minimize voids, segregation and "honey combing". This is a potential problem between the rebar mat and the face of the bulkhead forms. The 1/2-inch maximum aggregate size also enhances pneumatic transport. The fly ash is sufficiently fine grained that it does not occupy space in the mix, but fills voids that could otherwise be present in the concrete. Fly ash also decreases concrete permeability.

The mean 28-day concrete compressive strength, f'_{cr}, from tests on the typical 3000 psi bulkhead concrete mix should not be less than 4200 psi (ACI 318-95, Section 5.3.2.2 and Merritt, 1983, p 5-5). This f'_{cr} should conservatively yield the design concrete strength of 3000 psi, or higher, under the most adverse working conditions. This high mean strength is necessary because it is not possible to obtain the ACI specified number of compression test cylinder tests. The ACI specifies a minimum of 30 test sets, each set being the average of two tests from a concrete batch, to evaluate concrete mix strength (ACI 318-95, Sections 5.3.1.1 and 5.6.1.4 and ACI 214-77, Section 4.1). It is not possible to adjust the mix proportions to the specified 28-day compression test data in the field because it rarely takes one day to completely fill a bulkhead. Regardless, compression test cylinders of the concrete placed in the bulkheads should be prepared to verify the strength of the concrete. Curing of test specimens near the downstream bulkhead face is recommended because that is a more realistic bulkhead environment.

Bypass and Sampling Pipe Considerations

The corrosion resistance of bulkhead pipe penetrations must be considered with respect to bulkhead life. It would be best if there were no pipe penetrations. However, pipe penetrations are necessary to pass mine drainage through the bulkhead during construction. In addition, some means is necessary to permit release of impounded water, if required it some time in the future. It is also wise to be able to monitor water pressure behind the bulkhead in order to determine the elevation of the mine pool and any unanticipated impoundment loss preventing planned design head being achieved.

The corrosion rates and the resulting probable life of piping of various stainless steels and pipe diameters should be evaluated. This analysis must use the site specific acid mine water concentration and temperature. For example, the maximum measured surface corrosion rate (C_r) for the Carpenter 20Cb3 stainless steel pipe used at the Friday Louden Bulkhead was less than 0.005 in/yr for continuous exposure to the maximum solution concentration at the maximum pressure and temperature. The testing was performed by the pipe manufacturer.

The design problem is to estimate when the bursting strength of the corroded stainless steel piping will drop below the maximum hydraulic pressure ( ). The calculation method used for the Friday Louden pipe penetrations is shown in Equations 26 and 27.

$$P = \frac{2St}{D}$$

$$t_1 = \frac{PD}{2S} \qquad (26)$$

$$t_1 = \frac{PD}{2S} \quad (26)$$

P = maximum allowable hydraulic pressure (psi)
S = design fiber stress (psi)
2 for two pipe walls
t_1 = required minimum wall thickness (in)
t_2 = manufactured wall thickness (in)
D = nominal inside pipe diameter (in)
C_r = corrosion rate (in/yr)

The minimum estimated pipe life (Y) in years is:

$$Y = \frac{t_2 - t_1}{C_r} \quad (27)$$

The predicted life of the piping is conservative because the concentration of corroding chemicals should decrease over time.

The piping should also contain waterstops to positively prevent the acid mine water from moving along the outer surface of the bypass pipe. The waterstops can be provided by thrust rings to eliminate any reliance on skin friction to prevent the bypass pipe from being ejected from the bulkhead. Figure 4 indicates the combination of thrust rings and waterstops installed in the American Tunnel Bulkhead. The individual stainless steel thrust rings on the 12-inch inside diameter bypass pipe was designed to resist the total thrust of 76,000 lbs from the maximum possible head.

Earthquake Resistant Design

Acid mine drainage bulkheads should be checked for loading resulting from the maximum credible earthquake. The American Concrete Institute provides (ACI 318-95, Sections 9.2.2, 9.2.3, 9.2.5) the basis for evaluating earthquake loading. The total design load is defined by different load factors, as follows:

$$U = 1.05F + 1.40E \quad (28)$$

U = design load
F = fluid load
E = earthquake load

The fluid load is the maximum water head acting on the bulkhead. The earthquake load is defined by the acceleration of the water impounded along the line-of-sight behind the bulkhead plus the bulkhead itself. Figure 5 provides the Ransom Tunnel plan, showing the anticipated 360-foot line-of-sight water impounded upstream from the bulkhead. The calculations the Ransom Tunnel Bulkhead earthquake loading factor of safety are presented in Appendix A.

SUMMARY

Near surface concrete tunnel bulkheads have successfully impounded water, as indicated in Appendix B. Bulkheads can be safely designed to impound acid mine water by considering

1) the leakage potential along the concrete/rock contact,

2) the shear stress developed in the concrete and the rock,

3) the bending moment resistance of the bulkhead,

4) the hydrofracturing potential,

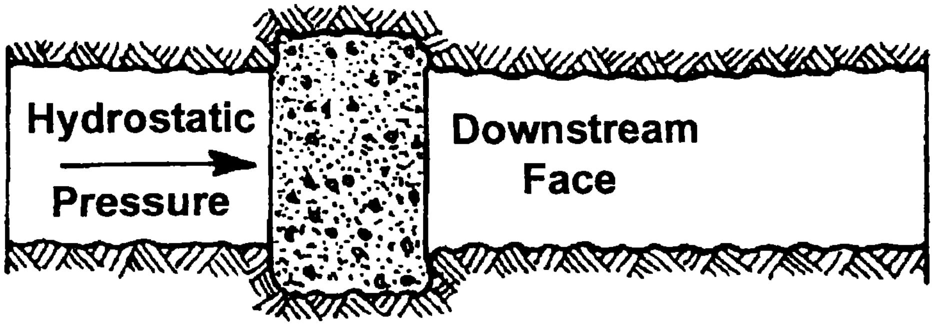
5) the corrosion rates for the piping and concrete and

6) the earthquake for the bulkhead area.

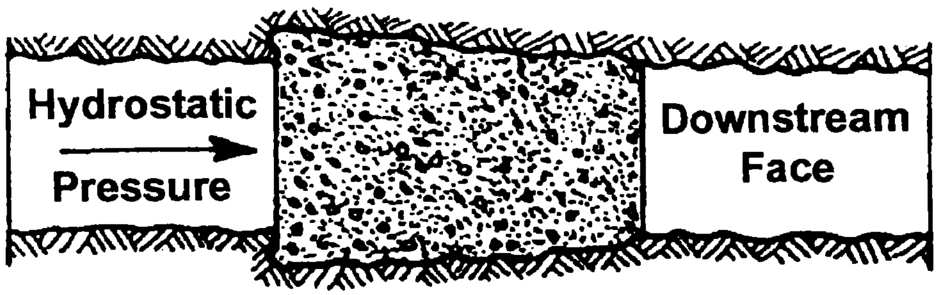
Of the 22 bulkheads that I have worked on, five did not successfully retain water. Three of those bulkheads were unsuccessful because of construction deficiencies and two because of unanticipated geologic conditions. One of the construction problems was repaired and one of the geologically deficient bulkheads redesigned and rebuilt to conform with the geology encountered.

Low-pressure grouting of the concrete/rock contact is recommended to increase the hydraulic gradient resistance between the plug and the rock and to decrease the required length of the bulkhead. The shear strength of the concrete and the rock must exceed the perimeter shear stress developed by the maximum head. The bending stress at the downstream face must either be kept below allowable plain concrete design tensile strength or steel tensile reinforcement must he placed near the down stream face to support the potential tensile stresses. The bulkhead must be installed at a depth sufficient to prevent hydrofracing the formation and the loss of acid water to the formation joint system. Chemical attack of the bulkhead concrete must be resisted and the corrosion rate of the piping through the bulkhead must be balanced by sufficiently thick pipe walls to provide for the required minimum bulkhead life. The site specific earthquake loading hazard should be evaluated.

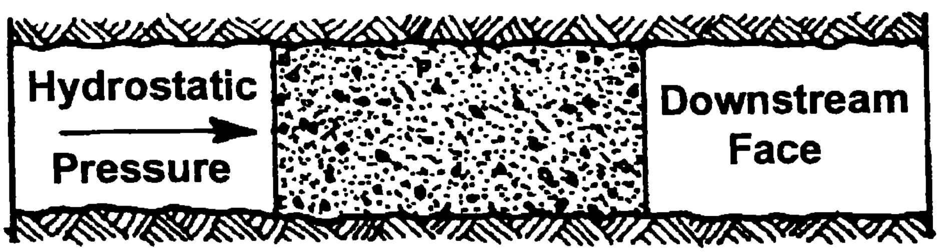
Figure 1. Types of bulkheads in use.



Slab keyed into walls



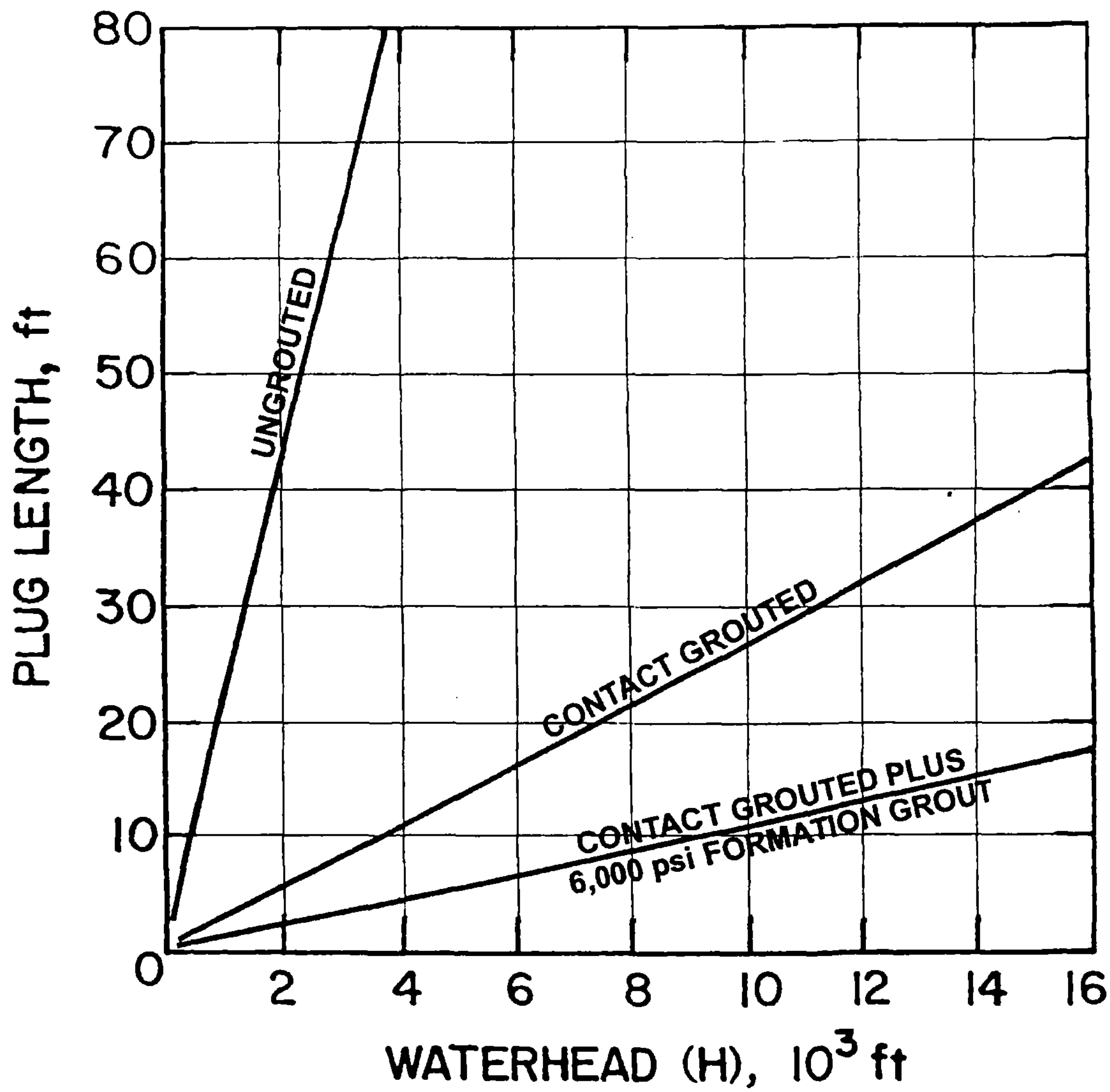
Taper plug



Parallel plug

(Adapted from Garrett & Campbell Pitt, 1958)

Figure 2. Test results from experimental bulkhead (Garrett & Campbell Pitt, 1958).



Adapted from Garrett and Campbell Pitt's 1958 test results for experimental bulkhead, Factors of Safety of 1.00

Figure 3. Compressive strength results, Chandler Tunnel.

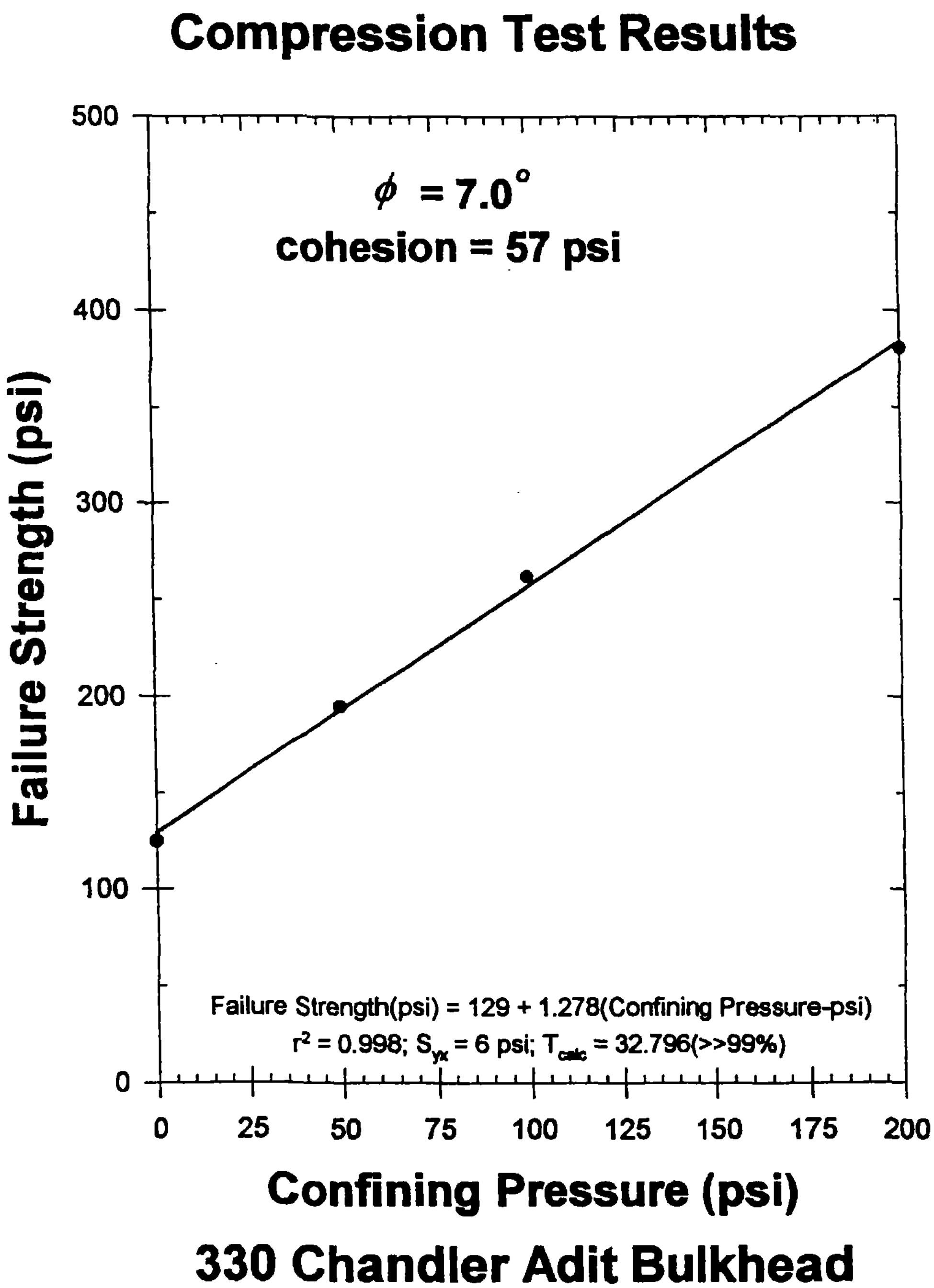


Table 1. Concrete selection based on sulfate concentration (ACI 318-95, Section 4.3.1)

TABLE 4.3.1—REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Maximum water-cementitious materials ratio, by weight, normal weight aggregate concrete*	Minimum f_c', normal weight and lightweight aggregate concrete, psi*
Negligible	0.00-0.10	0-150	—	—	—
Moderate†	0.10-0.20	150-1500	II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	4000
Severe	0.20-2.00	1500-10,000	V	0.45	4500
Very severe	Over 2.00	Over 10,000	V plus pozzolan‡	0.45	4500

* A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 4.2.2).

† Seawater.

‡ Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Figure 4. Longitudinal cross section of American Tunnel Bulkhead.

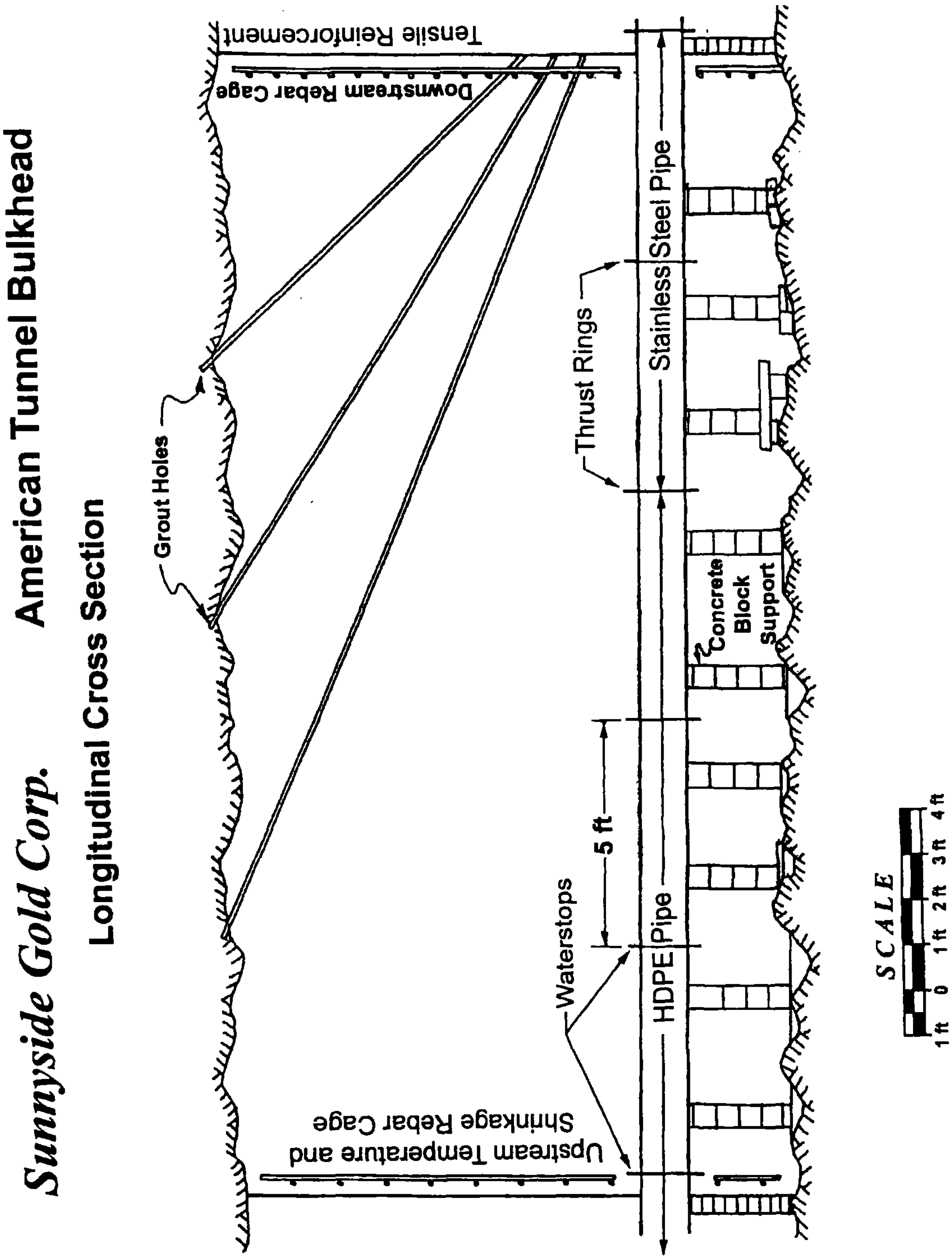
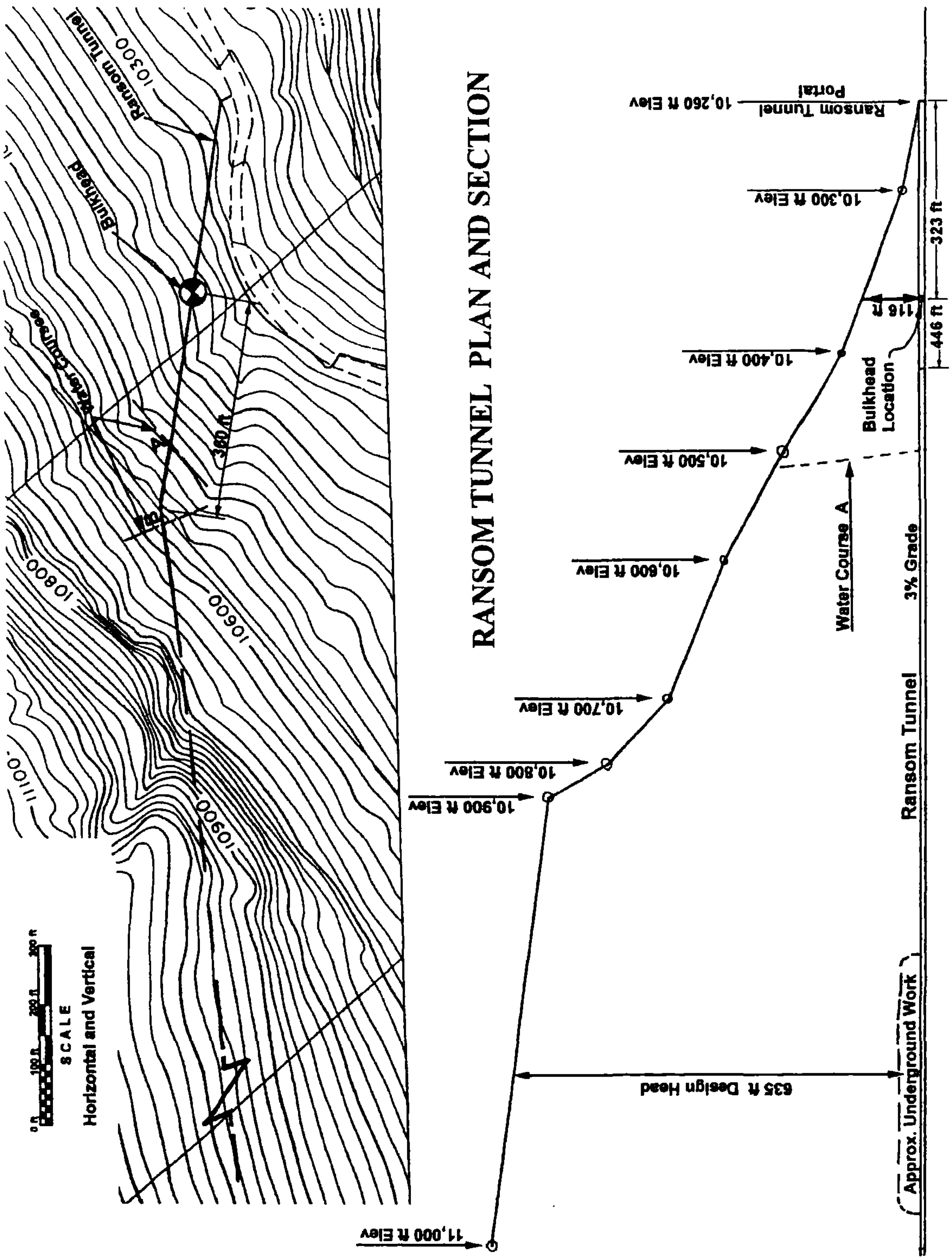


Figure 5. Ransom Tunnel Bulkhead location.



REFERENCES

Abel, J.F., Jr., 1993a, Bulkhead design for the Sunnyside Mine: consulting report to Sunnyside Gold Corp., 54 p

Abel, J.F., Jr., 1993b, Bulkhead design philosophy: consulting report to Intermountain Mine Services, Inc., 12 p

Abel, J.F., Jr., 1997, Bulkhead design for the Ransom Tunnel: consulting report to Sunnyside Gold Corp., 3 p

American Concrete Institute, 1977, Recommended practice for evaluation of strength test results of concrete (ACI 214-77): 23 p

American Concrete Institute, 1983, Recommended practice for evaluation of strength test results of concrete (ACI 214-77)(Reapproved 1983): 23 p

American Concrete Institute, 1989, Building code requirements for structural plain concrete (ACI 318.1-89) and commentary - ACI 318.1R-89: 14 p

American Concrete Institute, 1993, Guide for selecting proportions for high-strength concrete with Portland cement and fly ash (ACI 211.4R-93): 13 p

American Concrete Institute, 1995, Building code requirements for structural concrete (ACI 318-95) and commentary - ACI 318R-95: 369 p

American Institute of Steel Construction, 1989, Manual of steel construction, allowable stress design: 9th ed, AISC, Inc., 1121 p

J.D. Bredehoeft, R.G. Wolff, W.S. Keys & E. Shuter, 1976, Hydraulic fracturing to determine the regional in situ stress field, Piceance Basin: Colorado: GSA Bulletin, v 87, p 250-258.

Brown, A., Consultants, Inc., 1992a, Preliminary design of plug for the Reynolds Adit, Summitville Mine, Colorado: Report 1288B/920825, Aug 19, 33 p

Brown, A., Consultants, Inc., 1992b, Adit plugging case histories: Report 1288B/921105, Nov 5, 17 p

Brown, A., Consultants, Inc., 1992c, Model evaluation of the effectiveness of plugging the Reynolds Adit: Report 1288B/920813/R1, Nov 19, 26 p

REFERENCES (Continued)

Canadian Mining Jour, 1985, Anatomy of a crisis: Dec, p 10-11

Chekan, G.J., 1985, Design of bulkheads for controlling water in underground mines: USBM Info Circ 9020, 36 p

Coogan, J. & F.C. Kintzer, 1987, Tunnel plug design at Tyee Lake: Bulletin Assoc. Engineering Geologists, v 24, no 1, p 27-42

Einarson, D.S. & J.F. Abel, Jr., 1990, Tunnel Bulkheads for Acid Mine Drainage: Proc Symp on Unique Underground Structures, v 2, p. 71-1 to 71-20

Garrett, W.S. & L.T. Campbell Pitt, 1958, Tests on an experimental underground bulkhead for high pressures: Jour S. African Inst Mining and Metallurgy Congress, Oct, p 123-143

Garrett, W.S. & L.T. Campbell Pitt, 1961, Design and construction of underground bulkheads and water barriers: in Proc 7th Commonwealth Mining and Metallurgy Congress, v 3, p 1283-1301

Lancaster, F.H., 1964, Research into underground plugs Transvall and Orange Free State Chamber of Mines: Research Report 27/64, Aug, 130 p (Govt Mining Engr of S. Africa)

Lindeburg, M.R., 1989, Civil engineering reference manual: fifth ed, Professional Publications, Inc., 646 p

Loofbourow, R.L., 1973, Groundwater and groundwater control: in SME Mining Engineering Handbook, Section 26.7.4, Underground bulkheads and plugs, p 2646-2648

Louw, A., 1970, Ordeal by water: Mining Congress Jour, Part 1, Mar, p 43-56, Part 2, Apr, p 99-103

Merritt, F.S., 1983, Standard handbook for civil engineers: McGraw-Hill, 1578 p

Neukirchner, R.J. & D.R. Hinrichs, 1998, Effect of ore body innundation - a case study: Technical report provided by Eagle Engineering Services, Inc., 12 p

Obert, L. & W.I. Duvall, 1950, Generation and propagation of strain waves in rock: Part I, USBM Rpt of Investigation 4683

Petykopf, B.T., T.C. Atchison & W.I. Duvall, 1961, Photographic observation of quarry blasting: USBM Rpt of Investigation 5849

REFERENCES (Continued)

Singh, M.M., 1992, Mine subsidence: in SME Mining Engineering Handbook, ed 2, Chapt 10.6, Section 10.6.4.5 Hydrologic effects & Section 10.6.4.6 Nonmining damage, p 961-964

Troxell, G.E., H.E. Davis & J.W. Kelly, 1968, Composition and properties of concrete: 2nd ed, McGraw-Hill, 529 p

U.S. Bureau of Mines, 1992, Passive mine drainage treatment systems: Technology News, No 407A, AML #12A (reissue), 4 p

World Mining, 1969, How West Driefontein gold mine fought and won the flood battle: US Edition, Mar, v 5, no 3, p 18-23 & 36

Appendix A. Ransom Tunnel Bulkhead design calculations

Notation:

a = compression zone depth(in) minimum to balance rebar tension

A_s = area of rebar

b = beam width (1 in)

b_w = web width (12 in)

Bp = formation breakdown pressure (psi)

C = comp bending force (lb)

c = centroidal distance (in)

D = dead load $\left(\frac{lb}{ft}\right)$

d = distance, extreme compression fiber to rebar centroid (in)

E = earthquake load $\left(\frac{lb}{ft}\right)$

E_m = earthquake mass $\left(\frac{lb\text{-}sec^2}{ft}\right)$

ΣE = total earthquake load (lb)

F = fluid load $\left(\frac{lb}{ft}\right)$

FS = factor of safety

f'_c = concrete comp strength (3,000 psi)

$\sqrt{f'_c}$ = square root of f'_c

f_{ct} = concrete tensile strength $\left(5\phi\sqrt{f'_c}\ \text{psi}\right)$

f'_s = concrete shear strength (psi)

fy = rebar yield strength (60,000 psi)

g = acceleration due to gravity $\left(32.2\ \frac{ft}{sec^2}\right)$

H = design water head (635 ft)

h = tunnel height (10 ft)

I = moment of inertia

$K = \left(3.5 - 2.5\frac{M_u}{d}\right)$

= tunnel width (10 ft)

L = beam length or depth (10 ft)

M = bending moment (ft·lb)

M_n = nominal beam moment (ft·lb)

M_u = factored beam moment (ft·lb)

M_{ua} = earthquake beam moment (ft·lb)

m_c = minimum cover, form face to rebar surface (3.5 in)

S = section modulus (in^3)

S_l = line-of-sight distance (360 ft)

T = tensile bending force (lb)

U = required strength $\left(\frac{lb}{ft}\right)$

U_a = earthquake required strength

V_c = concrete shear strength (lb)

V_n = nominal shear force (lb)

V_s = rebar shear strength (lb)

V_u = factored shear force (lb)

v_s = rebar shear stress (psi)

W = bulkhead load (lb)

ω = uniform load $\left(\frac{lb}{ft}\right)$

α = earthquake acceleration $\left(0.087\frac{ft}{sec^2}\right)$

ρ = pressure head (275 psi)

ρ_g = pressure gradient $\left(\frac{psi}{ft}\right)$

$\rho_w = \frac{A_s}{b_w d}$

ϕ = strength reduction factors

0.90 flexure rebar tension

0.85 concrete shear

0.65 plain concrete flexure

γ_w = water density (62.4PCF)

γ_c = concrete density (151PCF)

γ_r = rock density (173 PCF)

σ_s = flexure stress (psi)

ω = uniform bulkhead load $\left(39{,}600\ \frac{lb}{ft}\right)$

Z = bulkhead design depth (ft)

Load factors (ACI 318, Sec 9.2.2, 9.2.3, 9.2.5)

Static fluid load factor (F) = 1.4;
Factor for fluid load under earthquake acceleration (F) = 1.05;
Earthquake accelerated load factor (E) = 1.40

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

Hydraulic pressure gradient:

Low pressure grouting of concrete-rock contact but not rock, gradient allowable = 41 psi/ft (Garrett & Campbell-Pitt, 1958, Chekan, 1985, p11), with factor of safety of 4

Ransom tunnel bulkhead, maximum pressure head

$$\rho = \frac{H\gamma_w}{144} = \frac{635(62.4)}{144} = 275 \text{ psi}$$

Required bulkhead length with low pressure grouting on concrete/rock bulkhead contact:

$$\mathbf{L = \frac{\rho}{40} = \frac{275}{40} = 6.9 \text{ ft}}$$

Pressure gradient with L = 8 ft $\quad \rho_8 = \frac{\rho}{8} = \frac{275}{8} = 34.4$ psi/ft

Factor of Safety against water leakage along concrete/rock contact around 8-ft thick bulkhead is:

$$\underline{\mathbf{FS}} = \frac{41}{34.4} = \underline{\mathbf{1.19}}$$

Concrete shear on Ransom tunnel perimeter:

$$f'_s = 2\sqrt{f'_c} = 2\sqrt{3000} = 110 \text{ psi} \qquad \text{(ACI 318-95, Sec 11.3.1.1)}$$

$$\mathbf{L} = \frac{\rho h l}{2(h+H)f'_s} = \frac{275(10)10}{2(10+10)110} = \frac{27500}{4400} = \mathbf{6.25 \text{ ft}}$$

$$W = \rho h = 275(10)10(144) = 3{,}960{,}000 \text{ lb}$$

$$v_s = \frac{W}{[2(h+H)]L(144)} = \frac{3960000}{[2(10+10)]8(144)} = 85.9 \text{ psi}$$

$$\underline{\mathbf{FS}} = \frac{f'_s}{v_s} = \frac{110}{85.9} = \underline{\mathbf{1.28}}$$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

Plain concrete deep beam bending stress design, Ransom tunnel (ACI 318-95, Sec 9.9.2.5, 18.4.1(b), & ACI 318-71, Sec 9.2.1.5)

Ransom Tunnel bulkhead, for 635-ft hydraulic head (275 psi pressure head):

$$\omega = U = 1.4\rho(144) = 1.4(275)144 = 55{,}400 \left(\frac{lb}{ft}\right)$$

$$M_n = \frac{\omega l^2}{8} = \frac{55400(10^2)}{8} = 692{,}500 \text{ ft·lb}$$

$$M_u = \frac{M}{5} \quad \frac{925}{.5} = 1{,}065{,}000 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bL^3}{12}}{\frac{L}{2}} = \frac{\frac{1(L^3)(12^3)}{12}}{\frac{L(12)}{2}} = \frac{144L^2}{6}$$

$$f'_{ct} = 3\sqrt{f'_c} = 3\sqrt{3000} = 164 \text{ psi}$$

$$f'_{ct} = 164 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{1065000}{\frac{144L^2}{6}} = \frac{44400}{L^2}$$

$L = \sqrt{\frac{44400}{164}} = \sqrt{271} =$ **16.5 ft**, length required for plain concrete bulkhead.

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144L^2}{6}} = \frac{1065000}{\frac{144(8^2)}{6}} = \frac{1065000}{1536} = 693 \text{ psi}$$

$$\mathbf{FS} = \frac{f'_{ct}}{\sigma_s} = \frac{164}{693} = \mathbf{0.24}$$

Therefore, 8-ft long plug must be reinforced.

Reinforced concrete deep-beam bending stress design, Ransom tunnel (ACI 318-95, Sec 9.3.2.3, Sec 9.3.2.3.: Wang & Salmon, 1985: Einarson & Abel, 1990)

$$C = \phi f'_c b_w = 0.85(3000)12a = 30600a$$

$$T = A_s f_y = 60000A_s$$

$$C = T; \qquad 30600a = 60000A_s; \qquad a = \frac{60000A_s}{30600} = 1.961A_s$$

$$M_n = \frac{\omega l^2}{8} = \frac{55400(10^2)}{8} = 692{,}500 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.9} = \frac{692500}{0.9} = 769{,}400 \text{ ft·lb } (9{,}233{,}000 \text{ in·lb})$$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

$M_u = A_s f_y(d - \frac{a}{2})$; $d = L - m_c = 8(12) - 3.5 = 92.5$ in

$M_u = 60000A_s(d - \frac{a}{2}) = 60000A_s(92.5 - \frac{1.961A_s}{2}) = 5550000A_s - 58800A_s^2$

Therefore: $9{,}233{,}000 = 5{,}550{,}000A_s - 58{,}000A_s^2$

$58{,}800A_s^2 - 5{,}550{,}000A_s + 9{,}233{,}000 = 0$

As = 1.69 $\frac{in^2}{ft}$ steel area required

#10 bars (1.270 in^2 per bar) on 8-in c-c provides 1.905 $\frac{in^2}{ft}$ steel area

Check for adequacy

Allowable $M_u = -58{,}800A_s^2 + 5{,}550{,}0000A_s = 10{,}360{,}000$ in·lb

Design Mu = 9,233,000 in·lb

$\underline{FS} = \frac{10360000}{9233000} = \underline{1.12}$

Critical section shear strength for Ransom tunnel, 8-ft deep beam bulkhead

Deep beam defined as $\frac{\ell}{d} < 5$ (ACI 318-95, Sec 11.8.1). Critical section shear at 0.15l (1.28 ft) from ribside (ACI 318-95, Sec 11.8.5), with #10 bars on 8-in c-c, there will be 1.905 $\frac{in^2}{ft}$ of steel per ft of beam width, d = 92.5 in (7.71 ft).

Detailed shear strength at critical section (ACI 318-95, Sec 11.8.7)

$\frac{\ell}{d} = \frac{10(12)}{[8(12)-3.5]} = \frac{120}{92.5} = 1.30 < 5$ Therefore, reinforced concrete bulkhead is a deep beam for design!

v_n - nominal shear stress shall not be greater than $8\sqrt{f'_c}$ when $\frac{\ell}{d} < 2$
(ACI 318-95, Sec 11.8.4)

Limiting value: $v_n \le 8\sqrt{3000} \le 438$ psi $V_n \le (v_n)b_w d \le (438)12(92.5) \le 486{,}200$ lb

$V_n = \frac{\omega\ell}{2} - (\frac{\omega\ell}{2})(\frac{0.15\ell}{0.5\ell}) = 0.35\omega = 0.35(55400)10 = 193{,}900$ lb

$V_u = \frac{V_n}{0.85} = \frac{193900}{0.85} = 228{,}100$ lb

$M_n = (\frac{\ell}{2})(0.15\,\ell) - \omega(0.15\,\ell)\frac{0.15\,\ell}{2} = 0.06375\frac{\omega\ell^2}{2} = 0.06375\frac{[55400(10^2)]}{2}$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

$M_n = 176{,}100$ ft·lb

$M_u = \frac{M_n}{0.9} = \frac{176100}{0.9} = 196{,}200$ ft·lb

$V_c = K\left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right)b_w d$

$K = 3.5 - 2.5\frac{M_u}{V_u d} = 3.5 - 2.5\left[\frac{196300}{228000\left(\frac{92.5}{12}\right)}\right] = 3.5 - 0.28 = 3.22$

K cannot exceed 2.5 Therefore K = 2.5

$\rho_w = \frac{A_s}{b_w d} = \frac{1.905}{(12)92.5} = 0.001716$ Trial, #10 bars on 8-in centers, two-way

$V_c = K\left(1.9\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right)b_w d$

$V_c = 2.5\left[1.9\sqrt{3000} + 2500(0.001716)\frac{228100\left(\frac{92.5}{12}\right)}{196200}\right]12(92.5)$

$V_c = 2.5[104.1 + 38.45]1110 = 2.5[142.55]1110 = 395{,}500$ lb

Allowable $Vc \le \left(6\sqrt{f_c'}\right)b_w d \le (6\sqrt{3000})12(92.5) = 364{,}800$ lb (ACI 318-95, Sec 11.8.7)

Therefore, **FS** $= \frac{V_c}{V_u} = \frac{364800}{228100} = \underline{\mathbf{1.60}}$

Ransom Tunnel bulkhead depth below surface (Z) required to prevent hydrofrac of rock around tunnel by 635-ft hydraulic head (275 psi pressure head): (Einarson & Abel, 1990)

$B_p = 3\sigma_{min} - \sigma_{max} = 2\sigma_{ovb} = 2Z\left(\frac{\gamma_r}{144}\right) = 2Z\left(\frac{173}{144}\right) = 2.403Z \text{ psi} = 275 \text{ psi}$

$\underline{\mathbf{Z}} = \frac{275}{2.403} = \underline{\mathbf{114\ ft}}$

Therefore, the bulkhead must be centered at least 320 ft inside the portal to develop 114 ft of overburden. Recommended bulkhead location from 319 ft to 327 ft inside the portal, for average distance from the portal of 323 ft and an average depth of 116 ft.

Earthquake bulkhead design; Load factors (ACI 318-95, Sec 9.2.2, 9.2.3, 9.2.5) Factor for fluid load under earthquake acceleration (F) = 1.05; Load factor for earthquake accelerated mass (E) = 1.40. Maximum credible earthquake acceleration (a) is 0.087 $\frac{ft}{sec^2}$.

$U = 1.05F + 1.40E$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

Mass (E) accelerated by maximum credible earthquake

$$E_m = \frac{S_l \gamma_w h\ell + L h \ell \gamma_c}{g} = \frac{[360(62.4)10(10)+8(10)10(151)]}{32.2} = \frac{[2{,}246{,}400+120{,}800]}{32.2} = 73{,}520\ \frac{\text{lb-sec}^2}{\text{ft}}$$

$$\Sigma E_m = E_m a = 73{,}520(0.087) = 6396\ \text{lb}$$

$$E = \frac{\Sigma E_m}{h} = \frac{6396}{10} = 640 \frac{\text{lb}}{\text{ft}}$$

Total load under earthquake acceleration

Ransom Tunnel bulkhead, for 635-ft hydraulic head:

$$\rho = \frac{H\gamma_w}{144} = \frac{635(62.4)}{144} = 275\ \text{psi}$$

$$F = \rho b_w(12) = 275(12)12 = 39{,}600\ \frac{\text{lb}}{\text{ft}}$$

$$U_a = 1.05F + 1.40E = 1.05(39600) + 1.40(640) = 41{,}580 + 896$$

$$U_a = 42{,}480\ \frac{\text{lb}}{\text{ft}}$$

Earthquake nominal beam bending moment

$$M_{na} = \frac{U_a \ell^2}{8} = \frac{42480(10^2)}{8} = 531{,}000\ \text{ft}\cdot\text{lb}$$

$$M_{ua} = \frac{M_n}{0.9} = \frac{531000}{0.9} = 590{,}000\ \text{ft}\cdot\text{lb}\ (7{,}080{,}000\ \text{in}\cdot\text{lb})$$

Steel area required for earthquake loading:

$$58800A_s^2 - 5{,}550{,}000A_s + 7{,}080{,}000 = 0$$

$A_s = 1.29\ \frac{\text{in}^2}{\text{ft}}$ Steel area required to resist maximum credible earthquake loading.

#10 bars on 8-in c-c provide 1.905 $\frac{\text{in}^2}{\text{ft}}$ steel area

Check for adequacy

Allowable $M_{ua} = -58{,}800A_s' + 5{,}550{,}000A_s = 10{,}360{,}000\ \text{in}\cdot\text{lb}$

Design $M_{ua} = 7{,}080{,}000\text{in}\cdot\text{lb}$

$\underline{FS} = \frac{10360000}{7080000} = \underline{1.46}$ against earthquake loading.

Appendix B. Some acid mine drainage bulkheads installed in Colorado

Mine/Location	Distance from Portal (ft)	Depth below Surface (ft)	Design Head (ft)	Years of Service	Comments
Eagle Mine, Gilman					Numerous seeps (9), 7 along Rock Creek, equilibrium water level 80 ft lower than design, poor quality initial water seeps, improved over time
Adit 6, '86	80	≈ 70	246	14	
Adit 5, '86	200	≈125	172	14	
Adit 7, '87	150	≈100	87	13	
Newhouse, '87	≈150	≈ 90	112	13	
Ben Butler Adit, '90	≈200	≈ 60	110	9	
Tip top Adit, '90	≈100	≈ 50	118	9	
Star of the West Incline, '90		≈130	101	9	Internal
Comet Claims, Placer Gulch, Silverton, '91					Initial <1.5 gpm leak Lower Level along fracture zone east side of plug, 1-in HDPE compression fitting on pressure gage line failed 2nd melt season
Lower Level	250	230	520	2	
Upper Level	150	122	295	2	
Thompson Creek Coal & Coke #1 Mine	30	20	Unk	< 8	Water spurting around thin (≈ 18-in), ungrouted plug
Sunnyside Gold Corp.					
American Tunnel	7950	2130	1550	4.5	1015-ft current head, ≈+30'/yr, pH 2.8 start, 5.4 now, w/20 tons lime placed, 5gpm initial leakage reduced to drips by regrouting
Terry Tunnel	3800	1160	650	3	≈ 109-ft current head, no leaks

Appendix B (Continued). Some acid mine drainage bulkheads installed in Colorado

Mine/Location	Distance from Portal (ft)	Depth below Surface (ft)	Design Head (ft)	Years of Service	Comments
Summitville Mine					
Reynolds Adit	1250	425	350	7	Minor dripping at downstream face, high strength alloy bolts severely corroded in 2 yrs, leakage through fracture system starting ≈ 100 ft downstream from bulkhead in 3120 psi rock
Chandler Adit	330	95	175	≈ 4	Initial 7-ft bulkhead failed at ≈ 85 ft head along 1-ft wide roof fault, overall 129 psi rock 20-ft extension, no leakage over ≈ 4 yrs

Appendix B (Continued). Some water impoundment bulkheads installed outside Colorado

Mine/Location	Distance from Portal (ft)	Depth below Surface (ft)	Design Head (ft)	Years of Service	Comments
Walker Mine, Plumas Cty, CA*	2700	810	500	13	Low permeability rock, maximum head 210 ft equilibrium at 120 ft of head, minor leakage
Mammoth Mine, Shasta Cty, CA*					
Friday Louden Tunnel	613	150	670	8	No leakage, ≲ 350-ft max head
Lower Gossan	200	100	300	7	No plug leakage, unknown head, pressure loss thru formation fractures
Upper Gossan	250	100	140	7	No leakage, unknown head
Keystone Mine, Shasta Cty, CA*					
Keystone 275	100	75	138	7	No leakage, unknown head
Keystone East Adit	400	250	288	7	No leakage, unknown head
Keystone 400 Level	200	100	450	0	No initial retention, 20-ft OD 130 psi grout ring added, failed to hold water. tight jointing (~2-in) weathered
Stowell Mine, Shasta Cty, CA*	200	200	300	7	Two portals w/plugs installed, No leakage, unknown head
Tyee Lake, AK Hydropower Tunnel	1500	790	1338	12	33gpm initial leakage, reduced to 11gpm by regrouting contact

* - Acid mine drainage bulkheads

Appendix B (Continued). Some historical worldwide records of bulkhead life

Mine	Length (ft)	Year Built	Design Head (ft)	Years of Service	Comments
CMR-6 Shaft 9 Level West, RSA	17	1953	830	45	Isolation bulkhead
East Daggerafontein 30 Haulage North and South, RSA	28	1949	1500	49	2 isolation bulkheads
Virginia 31 Haulage South, RSA	63	1957	3810	41	Isolation bulkhead, no leaks
Free State Geduld 47 Level, RSA	46	1955	1910	43	Emergency, 1st parallel sided bulkhead, full load in 72 hrs
Govt. G.M. Areas, RSA	5	1945	230	53	Isolation bulkhead
Sub Nigel, RSA	11	1953	459	45	Isolation bulkhead
West Dreifontein 10 & 12 Levels 4 bulkheads, RSA	60	1968	4000 (3740 actual)	30	Emergency 67,000 gpm inrush, pH 3.8, 2500 psi sand-concrete, alloy steel severely corroded 14-ft high, 12-ft wide
West Dreifontein	7.7	1958	15690	<1	Experimental bulkhead, 400 gpm leakage
Rocanville Mine, PCS Saskatchewan, CAN	87	1985	3000	13	Emergency 6,250 gpm inrush, potash mine from overlying aquifer, 8-ft high, 20-ft wide
Mammoth Mine, Shasta Cty, CA					
Friday Louden Tunnel	6	1980	212	≈1	Insufficient strength for 670-ft redesign head. Removed & rebuilt
Walker Mine, Plumas Cty, CA	15	1987	500	13	Maximum head 210-ft, minor leakage along contact and through formation

Detailed Work Plan and Benchmark Funding Schedule

#1 Send surface topography maps, tunnel long sections and any pertinent information to design engineer for guidance on probable plug location and plug size.

#2 Advance $15,000 per portal.
Open portal (portals if multiple levels) for 1 yd LHD access, establish ventilation and any other appropriate safety measures to secure portal and tunnel for selection of actual plug site and collecting rock samples. Build sediment traps as needed to control sludge that will be discharged.

#3 Close out #2 costs and advance $20,000.
Obtain Engineering design and submit to the Division of Minerals and Geology ("DMG") and the Water Quality Control Division ("Division") for approval.

#4 Establish coffer dam site, build coffer dam and divert water through piping.

#5 Close out #3 and #4 and advance $30,000.
Excavate plug area to solid rock, remove all loose rock and clean back, ribs and sill to remove mud, oxidation and other deposits to insure bonding of the concrete. Sand blasting works. Confirm size and taper assumptions used for design.

#6 Construct forms and place rebar. Arrange with DMG and the design engineer for pre-pour inspection. Determine grout pattern targets, mark hole collars to miss rebar and record drill angles and lengths to rock contact.

#7 Close out #5 and #6 and advance $10,000.
Place any alkaline material in the area between bulkhead and coffer dam planned for plug protection. Setup for pour and pour. Sample concrete for 7 day and 28 day tests during pour to confirm design strength has been met.

#8 Strip forms and drill holes for low pressure contact grouting. Grout holes until refusal.

#9 Close valve. Grout valve and close portal if permanent closure is selected by owner. Submit construction certification report to DMG and the Division. Close out #7, #8 and #9 and distribute Remaining Funds in accordance with the terms of this Agreement.

APPENDIX B
MOGUL MINE WORK PLAN

Remediation Plan: The owner has agreed to install a bulkhead in the tunnel of the Mogul No.1 Level to stop drainage to Cement Creek and Sunnyside Gold Corporation (SGC) has agreed to facilitate this project with a specified level of funding.

Remediating Party:
Gold King Mines Corporation
P.O. Box 299
Silverton, CO 81433

Contact: Stephen C. Fearn
President

Funding Party:
Sunnyside Gold Corporation
P.O.Box 177
Silverton, CO 81433

Contact: Larry Perino
Reclamation Manager

1. Description of Mining Activities

Physical Description of Conditions
The Mogul Mine No. 1 Level portal appears to discharge continuously. Water flowing from this portal carries dissolved metals. The regional geology is volcanic rocks with narrow veins containing base metals (Cd, Fe, Pb, Cu and Zn), which this tunnel was driven to intersect. There are limited known mine workings associated with this tunnel but it is connected physically to multiple levels of mine workings above this lower level that are more extensively mined. Sampling of waters from the portal has identified it as a major contributor of dissolved metal loading to Cement Creek.

General Description of the Mining Site
The history of the Mogul Mine is not known by SGC but it is believed to have been in operation in the early 1900's with some later activity occurring in the 1960's. The No.1 Level tunnel was probably driven around 1907 coinciding with the construction of the Mogul Mill at Gladstone. The No.1 Level portal is partially open but would require stabilization for safe access. The No.2 and No.3 Level portals are closed due to the collapse of the surface timber sets.

Identification of Lands
The Mogul No.1 Level is near Cement Creek approximately 1.5 miles above Gladstone in San Juan County, Colorado. See attached location map.

Identification of the Waters of the United States Potentially Affected
The headwaters of Cement Creek, Segment 7 of the Upper Animas River Basin.

2. Location Map
Attached

3. Stormwater Management Controls
Sediment catchments will be installed as needed. The majority of this project's activity will be underground.

4. Inspection and Record Keeping
The Reclamation Manager or a member of the Technical Services Department from SGC will inspect this project on a regular basis until project completion. Quarterly reports with photographs will be submitted by the remediating party to both the Water Quality Control Division ("Division") and the Colorado Division of Minerals and Geology("DMG"). Photographs of the property prior to remediation will be submitted with the first quarterly report. The contractor and/or Gold King is responsible for any inspection fees required by the DMG for inspections.

Monitoring
Additional monitoring for this project is not contemplated. SGC monitors Cement Creek above the American Tunnel complex monthly as a MLR Permit requirement. SGC will maintain this monitoring station until released from this requirement.

Reporting
The design will be submitted for approval by the Division prior to construction of the plug. Construction of the plug or plugs will not start until the design is approved by the Division. This is not a long-term project. Therefore, a final report will be submitted by the remediating party once all reclamation activities are complete as well as monthly progress reports while the project is active. Reports will be sent to DMG as well as the Division.

5. Mine Remediation Plan

Legal Right to Enter and Conduct Activities
Negotiations are in progress for Gold King Mines Corporation to be the owner of this property at the time the project is to be implemented.

If Gold King is removed as contractor under paragraph 11 of the underlying agreement then Gold King will allow SGC or its contractor to enter the property at its risk and place any plugs recommended by the design engineer.

Remedial Goals and Objectives

Reduction of metals loading to Cement Creek by removing the artificial drain created by the adit and reducing the exposure of metal bearing rock to oxygen and any chemical reactions this exposure may precipitate. The project is to be completed at the earliest feasible time after agreement(s) finalization but no later than September 30, 2003.

Site Loading Estimate

The site loading estimate is based on the mean loading from the Mogul Mine (from August 1999 to November 2001).
Based on this data, it is estimated that the average daily dissolved zinc loading for this site is 120 pounds per day.

Description of Project

The portal(s) of Number 1,2 and 3 levels will be opened as needed and studied for the best placement of plugs. In order for a plugs to be placed, a site meeting the following conditions will need to be found.

1) Location far enough underground to avoid the near surface fractures and joints caused by weathering.
2) Adequate rock compressive strength for structural stability.
3) A length of tunnel with minimal faulting or other geologic features.
4) Adequate ground cover over the potential site to resist the hydrostatic forces from the potential maximum head.
5) A location that eliminates or minimizes direct connections, stopes, raises or other openings to the overlying level.

If an acceptable location can be found a plug or plugs will be designed and installed. After installation is complete, the plug will be contact grouted. See attached paper on plugs (bulkheads).

Work Plan

1) Build catchments for potential sediment releases.
2) Operate the Cement Creek diversion and water treatment plant during de-watering.
3) Open and evaluate tunnel(s) for placement of plug(s).
4) Design and install plug(s).
5) Grout seal-rock contact.

Analysis

The plug proposed for the Mogul No. 1 Level will reduce the unsaturated zone by removing the drain. This will result in minimizing the oxygen available for reaction with the sulfide materials in the area. The hydrological conditions will be restored to an approximation of pre-mining conditions and should improve the water quality in the area.

Contingency Plans
Should the concept of plugs not be practical after engineering studies, the remediating party will consult with the DMG for other possible solutions. If an acceptable cost effective solution can be arrived at, such a system will be installed.

If after entry is gained, it is found that the source water could be better controlled by placing a plug in another level of the Mogul Mine, the plug location will be modified to gain the largest benefit from the project.

Monitoring
Additional monitoring for this project is not contemplated. SGC monitors Cement Creek above the American Tunnel complex monthly as a MLR Permit requirement. SGC will maintain this monitoring station until released from this requirement.

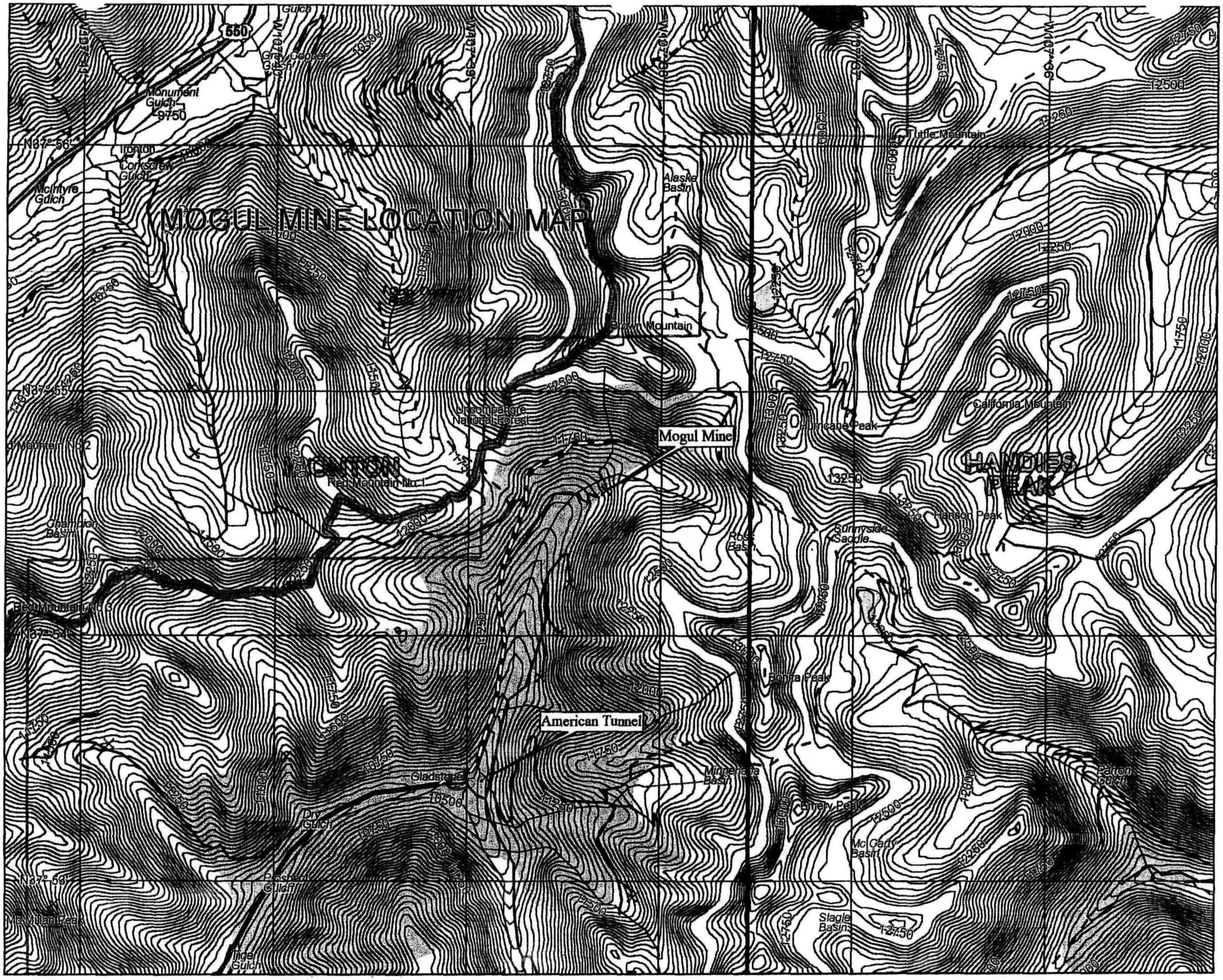
Budget
SGC will fund this project up to the total project limits defined in the Joint Petition for Fourth Amendment to Consent Decree to be executed by SGC and the Division. The anticipated level of funding is $300,000 which is believed to be adequate for at least two plugs if needed. SGC will also be funding up to $200,000 for plugging the Koehler Tunnel but the allocation can be adjusted as long as both projects are completed. The total funding level of $500,000 to complete the two projects is the maximum level of funding committed to by SGC.

Description of Land Use
This remediation work plan is intended to use Best Management Practices on the site to reduce metal loading to Cement Creek and to conform with land use policies for mining. The surface will not be materially changed as a result of the project and the bulkhead installation will not prevent future mining of the property with construction of facilities consistent with present day laws, land use policies and modern mining practices.

Consistency with Other Plans
The plan is consistent with the Animas River Stakeholders Group's plan to implement Best Management Practice projects in the Upper Animas River Basin to improve water quality and meet Water Quality Standards set as a goal. This property is on the list of projects identified that will need to be implemented to reach those goals.

Attachments:

Location Map
Bulkhead Design for Acid Mine Drainage
Detailed Work Plan and Benchmark Funding Schedule


MOGUL MINE LOCATION MAP
Mogul Mine
American Tunnel
Brown Mountain
Alaska Basin
Little Mountain
California Mountain
Hurricane Peak
HANDIES PEAK
Hanson Peak
Sunnyside Saddle
Bonita Peak
Emery Peak
Minnehaha Basin
Gladstone
Dry Gulch
Tiger Gulch
Champion Basin
Red Mountain No 1
Red Mountain No 3
Uncompahgre National Forest
Monument Gulch
Ironton
Corkscrew Gulch
McIntyre Gulch
Gray Copper Gulch
Slagle Basin
Mc Carty Basin
Ross Basin
Picayune Gulch
550

October 27-29, 1998

BULKHEAD DESIGN FOR ACID MINE DRAINAGE

John F. Abel, Jr.

in Proc Western U.S. Mining-Impacted Watersheds, Joint Conf on Remediation and Ecological Risk Assessment Technologies, Denver, CO

ABSTRACT

Impounding acid mine drainage behind a bulkhead in a mine tunnel has never been, and probably will never be, successful in reestablishing the pre-mining groundwater regime. However, even partially filling old mine workings should be beneficial. Partial filling should raise the mine depressed water table to the mine pool elevation. Partial filling of mine workings should decrease the quantity of groundwater entering mine workings, resulting in less mine drainage requiring treatment. Partial filling should deprive the submerged sulfide minerals of most of the oxygen necessary for producing acid, decreasing the rate of acid generation. Partial filling should improve the quality of acid water discharges from the mine. In effect, bulkheads can help but will never completely cure acid mine drainage.

Acid mine drainage bulkheads have several significant unknowns that potentially limit their usefulness:

1) What is the acceptable leakage around a tunnel bulkhead?

2) What are the natural flow paths for impounded acid mine water that may bypass a bulkhead into the open tunnel downstream from the bulkhead or to the ground surface?

3) How long will the bulkhead last?

4) Will unknown geologic conditions and(or) mine connections prevent the mine pool from reaching the planned elevation?

Concrete tunnel bulkheads designed to contain acid mine drainage water must be:

(1) long enough to prevent leakage along the contact between the concrete and the rock,

(2) thick enough to prevent shear failure in either the concrete or rock,

(3) prevent tensile failure of the downstream bulkhead face,

(4) deep enough to prevent hydrofracturing of the formation and

(5) acid resistant enough to last the requisite time.

The available design data includes, possibly in descending order of confidence, the strength and corrosion resistance of the concrete and steel, the strength of the rock, the maximum possible water head, the magnitude of the maximum credible earthquake and the in situ stress field.

INTRODUCTION

Ideally, impounding acid mine drainage behind a drainage tunnel bulkhead should reestablish the pre-mining groundwater regime. That hasn't happened and isn't likely to happen in the future. Even partially filling old mine workings should, however, be beneficial. Partial filling should raise the mine depressed water table to the mine pool elevation. Partial filling of mine workings should slow the rate and decrease the quantity of groundwater entering mine workings. Partial filling should deprive the submerged sulfide minerals of most of the oxygen necessary for producing acid, decreasing the rate of acid generation. In addition, partial filling should improve the quality of acid water discharges from the mine. In effect, bulkheads can help but will never be a complete cure for acid mine drainage.

Historically, and logically, mineral deposits have been exploited from the top down. This has resulted in many near surface access openings at the deposit outcrop. Some of the surface openings interconnect and some don't. Plugging the lowest draining portal may or may not significantly raise the level of the mine pool. Later in the life of a mine and a mining district, the deeper mine workings must be dewatered by pumping in order to continue mining. In such deeper mining operations, low level drainage tunnels may have been driven. Drainage tunnels have the potential, if plugged, for impounding water in a large part of the total mine excavation. Under no reasonable scenario, however, will plugging a single mine opening raise the mining depressed water table to its pre-mining level.

Acid mine drainage bulkheads have several significant unknowns that potentially limit their usefulness:

1) What is the acceptable leakage around a tunnel bulkhead, along the contact between the bulkhead and the rock and through the lower permeability rock immediately adjacent to the tunnel?

2) What are the natural flow paths for impounded acid mine water that may bypass a bulkhead into the tunnel downstream from the bulkhead or to the ground surface?

3) How long will the bulkhead last?

4) Will unknown geologic conditions and(or) mine connections prevent the mine pool from reaching the planned elevation?

Regardless of the location of a single bulkhead, water impounded upstream of the bulkhead may see the open downstream portion of the tunnel as a significant low resistance path for mine water discharge. The quantity of water forced back into the mine workings or to discharge at the ground surface by a bulkhead versus the quantity discharging into the downstream tunnel of a single bulkhead will depend on the rock substance, directional fracture and structure controlled permeability of the rock formation.

BULKHEAD DESIGN CONSIDERATIONS

Concrete tunnel bulkheads designed to contain acid mine drainage water must be:

(1) long enough to prevent leakage along the contact between the concrete and the rock,

(2) thick enough to prevent shear failure in either the concrete or rock,

(3) either thick enough to prevent tensile failure of the downstream face or contain sufficient tensile reinforcement to support the tensile stress,

(4) deep enough to prevent hydrofracturing of the formation,

(5) acid resistant enough to last the requisite time interval and

(6) strong enough to resist the maximum credible earthquake.

The available design data includes in descending order of confidence, the strength of the concrete and steel, if used, the strength of the maximum credible earthquake, the strength of the rock, the maximum possible water head and the in situ stress field.

Design of a concrete bulkhead can proceed once the mine layout and maximum possible hydraulic head are known and the bulkhead location selected on the basis of known hydrologic conditions and rock properties. The bulkhead location must first be prepared by removing rock loosened during the tunnel excavation.

Hydraulic Pressure Gradient

The pressure gradient (P_g) across a bulkhead is the hydraulic pressure, in psi, divided by the thickness of the bulkhead, in feet. Figure 1 presents the types of water-impoundment bulkheads generally used. It should be noted that the typical "taper plug", such as shown on Figure 1 is 7°. A bulkhead for a tunnel must be in intimate contact with the tunnel walls to prevent leakage along the concrete-rock interface around the plug. Bulkhead failure by leakage around the bulkhead, in the case of mine bulkheads, is more likely than failure of the bulkhead under thrust. Loofbourow in the Society of Mining Engineers (SME) Mining Engineering Handbook (1973, Sec 26.7.4) states "no indication of structural failure resulting from thrust was noted" in the case of ten bulkheads subjected by hydraulic pressures in excess of 1000 psi and which relied solely on normal rock surface irregularities, referred to as a "parallel plug" on Figure 1. High hydraulic pressure differentials across a bulkhead can be achieved by placing a long plug with a low resistance to water flow along the concrete-rock interface or by placing a short plug with high resistance to water flow along the concrete-rock interface achieved by grouting the concrete-rock contact. The Mining Engineering Handbook also recommends, in the same section, 40 to 25 feet of plug length for each 1000 psi of hydraulic head, i.e. pressure gradients from 25 to 40 psi/ft. The recommended concrete-rock grout pressure is "a few hundred psi". In practice, the grouting pressure must be kept below the formation breakdown pressure to prevent hydrofracturing. This limitation is particularly important for near surface bulkheads in order to prevent opening of fractures and possible release of impounded water through the formation to the open tunnel downstream or possibly even to the ground surface.

Garrett and Campbell Pitt (1961) reported the results from 26 mine bulkheads, 12 "parallel plugs", that relied solely on the irregularity of the tunnel walls, and 14 "taper plugs". However, they presented field data for 7 ungrouted bulkheads indicating acceptable leakage and pressure gradients from 18.0 to 26.2 psi/ft, averaging 21.4 psi/ft. The pressure gradient for the original ungrouted 6-ft thick bulkhead in the Friday Louden Tunnel was 15.3 psi/ft and did not leak when subjected to the measured 212 ft of head. Chekan (1985) analyzed Garrett and Campbell Pitt's pressure gradient data and produced a graphical version of their data. Figure 2 presents a modified version of the data that indicates that an ungrouted plug should be able to withstand a pressure gradient of approximately 21.3 psi/ft at a factor of safety of one. They also recommended a minimum factor of safety of 4 in good rock, yielding a recommended maximum pressure gradient of just over 5 psi/ft for average field conditions. Garrett and Campbell Pitt (1961) reported unacceptable leakage along the concrete-rock contact at 9.8 psi/ft when their ungrouted experimental bulkhead was pressurized to 75 psi. Obviously, the effectiveness of bulkhead concrete filling can vary widely, at least with respect to construction practice. It would not be realistic to attempt to build an ungrouted acid mine drainage bulkhead.

Garrett and Campbell Pitt indicated that pressure grouting of the concrete-rock contact of their experimental bulkhead would permit pressure gradients of 163 psi/ft without obvious leakage. Applying a factor of safety of four produces a design pressure gradient of over 40 psi/ft when the concrete-rock contact was grouted. The indicated benefit from pressure grouting the concrete-rock interface is an eight fold decrease in bulkhead length required to prevent unacceptable leakage.

What constitutes "unacceptable" leakage is a function of the bulkhead. The South African mining experience, reported by Garrett and Campbell Pitt (1961), indicates acceptable long term leakage along the concrete-rock contact and through the rock immediately around the bulkhead ranges from 3 gpm to 13 gpm and that 17 gpm was acceptable for short term leakage. Coogan and Kintzer (1987) indicate that 33 gpm leakage was not acceptable for a hydro tunnel but was acceptable when reduced to 11 gpm.

The leakage requirement for acid mine drainage bulkheads is generally more restrictive. In every case the goal is to reduce the flow to occasional drips at the bulkhead face. One contract specification is to limit the quantity of inflow at or within a specified distance from the downstream bulkhead face. The 1250 Bulkhead in the Reynolds Adit had such a requirement. The Reynolds Adit is in a weak, fractured and faulted rock formation. Before construction several tunnel sections were dripping measurable acidic groundwater. Limited formation grouting around the tunnel at the bulkhead location was employed before bulkhead construction. The purpose of the limited 6-foot radial formation grouting was to lower the permeability of the blast damage zone immediately adjacent to the tunnel walls. Obert and Duvall (1953) reported rock damage 48 hole radii from spherical explosive charges. Petykoph et al (1961) reported rock damage from 66 to 72 radii from cylindrical explosive charges. Since tunnel blasting always involves cylindrical charges the thickness of the blast damage zone was estimated as approximately 4.5 feet, in this case. Formation grouting beyond the potential blast damage zone was not undertaken because the specific fracture flow channels were not known. After water impoundment, groundwater inflow increased at a faulted tunnel section about 100 feet downstream from the bulkhead.

Garrett and Campbell Pitt indicated that high-pressure grouting of the rock adjacent to a bulkhead will result in a considerable increase in the allowable pressure gradient across the plug. However, high-pressure grouting is not an option for near surface plugging of old mine tunnels. Near surface high-pressure grouting could result in hydrofracturing of the rock around the tunnel.

The length (L) of a low-pressure grouted bulkhead necessary to meet the 40 psi/ft hydraulic pressure gradient criteria necessitates the calculation of the maximum pressure head (ρ), as follows:

$$\rho = \frac{H\gamma_w}{144} \qquad (1)$$

H - design water head
γ_w - water density

The required bulkhead length with low pressure grouting is:

$$L = \frac{\rho}{40} \text{ ft} \qquad (2)$$

Perimeter Shear Strength Design

Bulkhead design to resist shear stresses resulting from water impoundment involves evaluating concrete and rock shear strength along the perimeter of the tunnel and shear in the concrete at the critical section, as defined by the American Concrete Institute (ACI 318-95, Sections 11.8.1 and 11.8.5). Critical section shear includes the reinforcing bars, if present, in the designated section and, therefore, cannot be evaluated until the bulkhead reinforcing steel is tentatively selected.

The first requirement for evaluating bulkhead shear strength at the perimeter of the tunnel involves testing the rock to see whether the rock is stronger than the design concrete shear strength. Typically, the shear strength, cohesion, of the rock exceeds the design shear strength of the concrete. The measured compressive strength of the intact latite porphyry that is adjacent to the Ransom Tunnel bulkhead design example in Appendix A ranges from 10,260 psi to 35,570 psi and the estimated shear strength, cohesion, from approximately 2,500 psi to 8,900 psi. The concrete design shear strength (f'_s), for the 3,000 psi concrete compressive strength (f'_c) is 110 psi, specified by the American Concrete Institute as follows:

$$f'_s = 2\sqrt{f'_c} = 2\sqrt{3000} = 110 \text{ psi} \quad \text{(ACI 318-95, Sec 11.3.1.1)} \tag{3}$$

Obviously, the concrete is the critical design component for perimeter bulkhead shear at the Ransom Tunnel. This not always the case as was the case for the best ground in the Chandler Tunnel at the Summitville Mine, as shown on Figure 3.

When concrete design shear strength (f'_s) is less than the rock cohesion (c_r), the bulkhead length (L) needed to support the maximum perimeter shear stress from the application of the maximum pressure head (ρ), for a rectangular tunnel cross section with a height of (h) and width of (ℓ) is:

$$L = \frac{\rho h \ell}{2(h+\ell) f'_s} \text{ ft} \tag{4}$$

When rock cohesion (c_r) is less than the concrete design shear strength (f'_s), rock cohesion replaces concrete design shear strength in Equation 4.

Plain Concrete Deep Beam Bending Stress Design

The American Concrete Institute's "Building Code Requirements for Reinforced Concrete (ACI 318-95)" are recommended for design because the bulkheads are analogous to reinforced deep-beam concrete structures and because of the inherent conservatism of the code. It is difficult to obtain good adhesion between a concrete bulkhead and the roof and floor of a tunnel. The difficulty lies in completely cleaning of the floor and keeping it clear of mud and rock until the concrete is poured and in completely filling all the voids in the roof, even with low-pressure grouting. The deep-beam bulkhead should be conservatively assumed to act only one-way, between the walls (ribsides) of the tunnel. However, two-way reinforcing steel should be provided in bulkhead design to transfer some load to the tunnel roof and floor despite the difficulty in achieving intimate contact with the roof and removing all the loose rock from the floor. The one-way design assumption in effect produces a potential factor of safety of two, provided the more difficult roof and floor contacts between the bulkhead concrete and the rock are actually achieved by the recommended low-pressure contact grouting.

The recommended deep-beam bending analysis is based on a uniformly-loaded beam supported by the tunnel walls. This conservative design approach can be further justified by the inability of obtaining access to the upstream side of a bulkhead and the long life expected of the plugs.

The length of an unreinforced, plain, concrete bulkhead necessary must keep the tensile bending stresses in the downstream face below ACI allowable concrete tensile stress (f_t). ACI (318-95, Section 9.3.5 and 318.1-89, 1989, Section 6.2.2) directs that a strength reduction factor of 0.65 be used in design. ACI (1989, sec 6.2.1 and 318-95, Section 22.5.1) directs that the design tensile concrete bending stress not exceed:

$$f_t = 5\sqrt{f_c'} \tag{5}$$

f_c' = concrete design compressive strength

This amounts to 274 psi for 3,000 psi concrete. ACI (318-95, Section 9.2) also requires a 1.4 load factor for definable fluid loads.

The required length of an unreinforced plain concrete bulkhead to prevent tensile cracking on the downstream bulkhead face for a one-way (rib to rib) deep beam follows. The first step is to calculate the maximum nominal bending moment (M_n) on the one-way beam, as follows:

Fluid load per lb/ft^2

$$w = 1.4(p)144 \tag{6}$$

Maximum nominal bending moment

$$M_n = \frac{w\ell^2}{8} \text{ ft}\cdot\text{lb} \tag{7}$$

Nominal bending moment adjusted for capacity reduction factor (ϕ) of 0.65 to obtain the factored design bending moment (M_u :

$$M_u = \frac{M_n}{\phi} = \frac{M_n}{0.65} \text{ ft}\cdot\text{lb} \tag{8}$$

Maximum flexural stress

$$\sigma = \frac{M_u}{S} \text{ psi} \tag{9}$$

S = section modulus (in^3)

$$\text{Section modulus (in}^3) = \frac{I}{c} \tag{10}$$

I = moment of inertia (in^4)
c = centroidal distance (in)

$$\text{Moment of inertia (in}^4) = \frac{bL^3}{12} \qquad (11)$$

b = beam width (in)
L = beam depth (bulkhead length) (in)

$$\text{Centroidal distance (in)} = \frac{L}{2} \qquad (12)$$

Therefore, allowable flexural stress (f_t) in psi is

$$f_{cl} = \frac{M_u}{S} = \frac{M_u}{\frac{I}{c}} = \frac{M_u}{\left(\frac{\frac{bL^3}{12}}{\frac{L}{2}}\right)} = \frac{6M_u}{bL^2}$$

$$L^2 = \frac{6M_u}{b(f_t)} \qquad (13)$$

Required length (L) of plain concrete bulkhead, obtained by solving equation (13) for the beam depth (L), the bulkhead length, is presented for the Ransom Tunnel Bulkhead in Appendix A. The length of the bulkhead can be decreased by the use of tensile reinforcement, provided the plain concrete bulkhead is longer than needed to limit the hydraulic pressure gradient to an acceptable level. Normally a trial bulkhead length is selected and the reinforcement required to support the tensile bending stress calculated.

Reinforced Concrete Deep Beam Bending Stress Design

The length of an alternative reinforced plain concrete bulkhead depends on providing sufficient reinforcing steel to support the entire tensile bending stress developed in the deep beam bulkhead. The ACI capacity reduction factor for bending of a reinforced concrete deep beam (ACI 318-95, Section 9.3.2.1) is 0.90. The method employs the rectangular compressive stress distribution approximation. The ACI method is described in Section 10.2, for a reinforced simple concrete beam and Section 10.7 for deep beams. These ACI Sections define a simple, simply supported, deep beam as one whose depth exceeds 4/5 the span. ACI defines a reinforced continuous concrete beam as a deep beam if the depth exceeds 2/5 the span. A bulkhead that can rotate at its supports, a simply supported beam, would be unlikely to be able to retain a fluid. The recommended low-pressure grouting is designed to fix the roof, walls and floor of the bulkhead, preventing rotation. In the case of the 10-foot width of the Ransom Tunnel Bulkhead design in Appendix A, the 8-foot thick bulkhead is a deep beam for design in either case.

The compressive load toward the upstream (water side) face of the bulkhead must be balanced by the tensile reinforcement from the rebar cage a few inches from the downstream (air side) face. Deep beam design assumes that a uniform compressive stress equal to 0.85 times the specified concrete compressive strength acts over an area 1 ft wide by 0.85 times the centroidal distance in depth (a) below the loaded surface. The constant, 0.85, is reduced 0.05 for each 1,000 psi the concrete strength exceeds 4,000 psi. The method, as further described by Wang and Salmon (1985, p 43-44), assumes the tensile reinforcing steel yields before the concrete crushes under bending induced compressive stress. Tensile reinforcement design for the typical reinforced concrete deep beam bulkhead follows:

Compressive force $C = \phi(f'_c)ba = 0.85(f'_c)ba$ (14)

Tensile force $T = A_s f_y$ (15)

b = beam width (in.)
a = compression zone depth (in.)
A_s = steel area (sq in./ft)
f_y = steel yield stress (psi)
f'_c = concrete strength (psi)

The method presented by Wang and Salmon (1985) assumes that the compressively stressed concrete area is no deeper into the beam than necessary to carry the bending moment developed compressive force at the ACI specified compressive stress of 0.85 times the specified compressive strength. The calculations equating C to T using equations (14) and (15), using 3000 psi concrete and 60000 psi yield strength rebar follow:

$$C = T \quad 0.85(f'_c)ba = A_s f_y \quad 0.85(3000)12a = 60000A_s$$

$$a = \frac{A_s f_y}{0.85(f'_c)12} = \frac{60000A_s}{0.85(3000)12} = 1.96078A_s \tag{16}$$

Summation of moments about center of the compressively stressed area, substituting the compression zone depth from equation (16)

$$M_u = A_s f_y\left[d-\frac{a}{2}\right] = A_s f_y\left[(L-m_c)-\frac{a}{2}\right] \tag{17}$$

d = depth, top of beam to center of reinforcing steel (in) = $L-m_c$
M_u = factored design beam bending moment (in·lb)
m_c = minimum cover, form face to rebar surface (inches)

$$M_u = A_s f_y\left[(L-m_c)-\frac{a}{2}\right] = 60000A_s\left[(L-m_c)(12) - 1.96078\frac{A_s}{2}\right] \text{ in·lb}$$

$$M_u = 60000A_s[(L-m_c)12-0.98039A_s] \text{ in·lb} \qquad (18)$$

The design flexure resisting bending moment is:

$$M_u = \frac{M_n}{\phi} = 60000A_s[(L-m_c)12-0.98039A_s] \text{ in·lb} \qquad (19)$$

The solution for reinforcing steel area per foot of the one-way beam from rib to rib requires equating Equation 19 to the bending moment resulting from factored load. The nominal maximum bending moment is $M_n = \frac{w\ell^2}{8}$ ft·lb, as calculated by Equation 7. However, the strength factor (ϕ) for the steel tensile reinforcement is 0.90 and the factored bending moment from loading becomes:

$$M_u = \frac{M_n}{\phi} = \frac{\frac{\omega\ell^2}{8}}{0.90} = \frac{\omega\ell^2}{0.9(8)} \text{ ft·lb} \qquad (20)$$

Therefore, the minimum required steel area per foot (A_s) is calculated from equating Equation 19 and Equation 20.

The design bulkhead thickness typically required to prevent leakage due to the hydraulic pressure gradient and to resist the perimeter shear forces makes the use of a simple beam design for bending of a possibly fixed-end beam extremely conservative. The bending deformations causing appreciable reinforcing steel strain, and therefore tensile stress, will not be linear due to the bulkhead thickness and the lateral restraint provided by the tunnel ribs. Bulkhead failure would most likely occur by concrete yielding of a pressure arch that would develop in the upstream side, rather than as the result of yielding of the reinforcing steel. Reinforcing steel is required at both the downstream and upstream bulkhead faces to control temperature and shrinkage induced stresses in the large bulkhead pour.

Critical Section Shear

ACI requires evaluation of critical section shear if the ratio of the bulkhead span (ℓ) divided by the distance (d) from the upstream bulkhead face to the centroid of the reinforcing steel $\left[\frac{\ell}{d}\right]$ is less than 5 (ACI 318-95, Section 11.8.1). This appears to always be the case for bulkheads that meet the pressure gradient requirement. This evaluation is very complex and has not been critical to bulkhead design. An example of the critical section shear evaluation method is presented in Appendix A.

Bulkhead Depth Based on Hydraulic Pressure

Hydrofracturing, generally referred to as hydrofracing, of sedimentary formations from drillholes is frequently undertaken for the purpose of stimulating oil well production. Formation breakdown pressure (B_p) is a function of (1) the tensile strength of the rock immediately adjacent to the drillhole, (2) the in situ stress field in the plane perpendicular to the drillhole and (3) the pore pressure present in the formation. Bredehoeft, et al (1973) presented a study of drillhole hydrofracturing of a competent rock. They presented the following well known equation for breakdown pressure:

$$B_p = T_s + 3S_{min} - S_{max} - P_f \tag{21}$$

All terms in psi

T_s = tensile strength
S_{min} = minimum stress normal to the borehole
S_{max} = maximum stress normal to the borehole
P_f = formation pore pressure

The equation can be simplified for the case of hydraulic pressure behind an acid mine drainage bulkhead in a tunnel. First, the tensile strength can be assumed to be zero because the rock adjacent to a tunnel is jointed and generally damaged by blasting. The packed-off section of a drillhole, on the other hand, can be entirely within one joint block and is not subject to blasting damage. Second, the pore pressure present near surface and adjacent to a tunnel must be low and can also be assumed to be zero. Finally, in the absence of in situ stress measurements it is necessary to estimate the stresses in the plane normal to the tunnel. The simplest assumption is for hydrostatic stress conditions equal to the overburden stress. The assumption is generally conservative since the overburden stress must be present and the more general stress state measured is for near surface horizontal stresses to equal or exceed the overburden stress. Normal formation breakdown pressures encountered in shallow oil field work range from 1.4 to 2.8 times the overburden stress. This indicates that the hydrostatic stress assumption, where the formation breakdown (hydrofracing) pressure equals two times the overburden stress, is not unreasonable.

The resulting simplified breakdown pressure equation is:

$$B_p = 2S_{ovb} \tag{22}$$

S_{ovb} = overburden stress (psi)

Acid mine drainage bulkheads must be placed at a depth which will not result in hydrofracing the rock adjacent to the tunnel, i.e. opening of the joints and fractures and injection of acid mine water into the rock mass around the plug and possibly to the ground surface.

The hydraulic breakdown pressure (B_p) available to hydrofrac the rock immediately upstream from the plug and adjacent to the tunnel is the maximum potential head. Therefore, the overburden stress must be sufficient to prevent hydrofracing. The required overburden stress (S_{ovb}) is:

$$S_{ovb} = \frac{B_p}{2} \tag{23}$$

The overburden pressure is the product of the depth (H) and density (γ) of the overlying rock. Since the density can be readily measured, the depth of the bulkhead must be selected to limit the possibility of hydrofracing, as follows:

$$S_{ovb} = \frac{\gamma H}{144} = \frac{B_p}{2} \tag{24}$$

γ = density (PCF)
H = depth (ft)

$$H = \frac{72 B_p}{\gamma} \tag{25}$$

Corrosion Resistant Design

The useful life of a concrete bulkhead is controlled by the corrosive nature of the acid mine drainage being impounded, the formulation of the concrete mix and on the corrosion resistance of the piping penetrating through the bulkhead. The corrosion characteristics of the impounded acid mine drainage can not be controlled. It is likely that the quality of the drainage water will change during the course of mine filling and after the maximum head has been reached. Sampling of the water impounded immediately behind the American Tunnel bulkhead has shown wide pH fluctuations since the valve was closed. Initially the pH rose well above 8, apparently as the result of the 20 tons of lime and 20 tons of limestone placed upstream from the bulkhead. The pH has dropped to 2.8 in the last year possibly as the result of solutioning of precipitates that have accumulated on the walls of underground openings that have since been inundated. However, no iron has been detected in the water samples taken at the American Tunnel Bulkhead. Iron present as Fe^{+3} ions tend to surface coat limestone limiting its further dissolution (USBM, 1994).

Concrete Mix Considerations

Chemical attack on the bulkhead concrete exposed to sulfate concentrations in the impounded mine water in contact with a bulkhead can be resisted by using Type V, sulfate resistant cement, as required by the ACI (ACI 318-95, Section 4.3). Table 1 presents the ACI requirements for concrete exposed to various sulfate concentrations. Brown (1992b, p 1) indicated that the 1250 Bulkhead in the Reynolds Adit at the Summitville Mine would be subjected to a 4643 mg/l (ppm) sulfate ion concentration. In addition, pozzolan (fly ash) can be added to the concrete mix to decrease concrete permeability and improve sulfate resistance, as recommended by ACI (ACI 318-89, Table 4.3.1) and Troxell et al (1968, p 104) for concrete in contact with "Very Severe", greater than 10000 ppm, sulfate concentrations.

A typical 3000 psi bulkhead concrete mix is 1 sack of Type V cement (94 lbs), to 235 lbs of fine aggregate (sand) to 330 lbs of well graded coarse aggregate and 15 lbs of fly ash (pozzolan). The mix proportions are 1:2.5:3.5 (cement, sand, gravel). One yard of concrete would contain 5.7 bags of cement (536 lbs), 1340 lbs of sand, 1881 lbs of well graded 1/2-inch maximum coarse aggregate and 86 lbs of fly ash, pozzolan. One yard of the specified concrete would have a dry weight of 3843 lbs/yard and a mixed weight of 4085 lbs/yard when the required 29 gallons of water is added. See ACI 211.4R-93 for additional details. The approximate in-place density of the concrete will be 151 lb/cu ft.

The typical mix would normally be considered "oversanded". However, the higher than normal sand content is designed to increase pumpability, i.e. slump, at the low water/cement ratio of 0.45 required to resist "Severe" or "Very Severe" concentrations of sulfate in acid mine water. High slump concrete can be pumped as a wet mix through a slick-line or pneumatically blown as a dry mix with the water added as placed in the bulkhead as shotcrete. Pneumatic transport is possible over greater distances but with a more variable field controlled water/cement ratio.

The Standard Handbook for Civil Engineers (Merritt, 1983, Table 8-4) indicates that a well-graded aggregate with a maximum size of 2 inches can be used with the mix proportions specified. However, it is recommended that 1/2-inch maximum aggregate size be used to minimize voids, segregation and "honey combing". This is a potential problem between the rebar mat and the face of the bulkhead forms. The 1/2-inch maximum aggregate size also enhances pneumatic transport. The fly ash is sufficiently fine grained that it does not occupy space in the mix, but fills voids that could otherwise be present in the concrete. Fly ash also decreases concrete permeability.

The mean 28-day concrete compressive strength, f'_{cr}, from tests on the typical 3000 psi bulkhead concrete mix should not be less than 4200 psi (ACI 318-95, Section 5.3.2.2 and Merritt, 1983, p 5-5). This f'_{cr} should conservatively yield the design concrete strength of 3000 psi, or higher, under the most adverse working conditions. This high mean strength is necessary because it is not possible to obtain the ACI specified number of compression test cylinder tests. The ACI specifies a minimum of 30 test sets, each set being the average of two tests from a concrete batch, to evaluate concrete mix strength (ACI 318-95, Sections 5.3.1.1 and 5.6.1.4 and ACI 214-77, Section 4.1). It is not possible to adjust the mix proportions to the specified 28-day compression test data in the field because it rarely takes one day to completely fill a bulkhead. Regardless, compression test cylinders of the concrete placed in the bulkheads should be prepared to verify the strength of the concrete. Curing of test specimens near the downstream bulkhead face is recommended because that is a more realistic bulkhead environment.

Bypass and Sampling Pipe Considerations

The corrosion resistance of bulkhead pipe penetrations must be considered with respect to bulkhead life. It would be best if there were no pipe penetrations. However, pipe penetrations are necessary to pass mine drainage through the bulkhead during construction. In addition, some means is necessary to permit release of impounded water, if required it some time in the future. It is also wise to be able to monitor water pressure behind the bulkhead in order to determine the elevation of the mine pool and any unanticipated impoundment loss preventing planned design head being achieved.

The corrosion rates and the resulting probable life of piping of various stainless steels and pipe diameters should be evaluated. This analysis must use the site specific acid mine water concentration and temperature. For example, the maximum measured surface corrosion rate (C_r) for the Carpenter 20Cb3 stainless steel pipe used at the Friday Louden Bulkhead was less than 0.005 in/yr for continuous exposure to the maximum solution concentration at the maximum pressure and temperature. The testing was performed by the pipe manufacturer.

The design problem is to estimate when the bursting strength of the corroded stainless steel piping will drop below the maximum hydraulic pressure ( ). The calculation method used for the Friday Louden pipe penetrations is shown in Equations 26 and 27.

$$P = \frac{2St}{D}$$

$$t_1 = \frac{PD}{2S} \qquad (26)$$

$$t_1 = \frac{PD}{2S} \quad (26)$$

P = maximum allowable hydraulic pressure (psi)
S = design fiber stress (psi)
2 for two pipe walls
t_1 = required minimum wall thickness (in)
t_2 = manufactured wall thickness (in)
D = nominal inside pipe diameter (in)
C_r = corrosion rate (in/yr)

The minimum estimated pipe life (Y) in years is:

$$Y = \frac{t_2 - t_1}{C_r} \quad (27)$$

The predicted life of the piping is conservative because the concentration of corroding chemicals should decrease over time.

The piping should also contain waterstops to positively prevent the acid mine water from moving along the outer surface of the bypass pipe. The waterstops can be provided by thrust rings to eliminate any reliance on skin friction to prevent the bypass pipe from being ejected from the bulkhead. Figure 4 indicates the combination of thrust rings and waterstops installed in the American Tunnel Bulkhead. The individual stainless steel thrust rings on the 12-inch inside diameter bypass pipe was designed to resist the total thrust of 76,000 lbs from the maximum possible head.

Earthquake Resistant Design

Acid mine drainage bulkheads should be checked for loading resulting from the maximum credible earthquake. The American Concrete Institute provides (ACI 318-95, Sections 9.2.2, 9.2.3, 9.2.5) the basis for evaluating earthquake loading. The total design load is defined by different load factors, as follows:

$$U = 1.05F + 1.40E \quad (28)$$

U = design load
F = fluid load
E = earthquake load

The fluid load is the maximum water head acting on the bulkhead. The earthquake load is defined by the acceleration of the water impounded along the line-of-sight behind the bulkhead plus the bulkhead itself. Figure 5 provides the Ransom Tunnel plan, showing the anticipated 360-foot line-of-sight water impounded upstream from the bulkhead. The calculations the Ransom Tunnel Bulkhead earthquake loading factor of safety are presented in Appendix A.

SUMMARY

Near surface concrete tunnel bulkheads have successfully impounded water, as indicated in Appendix B. Bulkheads can be safely designed to impound acid mine water by considering

1) the leakage potential along the concrete/rock contact,

2) the shear stress developed in the concrete and the rock,

3) the bending moment resistance of the bulkhead,

4) the hydrofracturing potential,

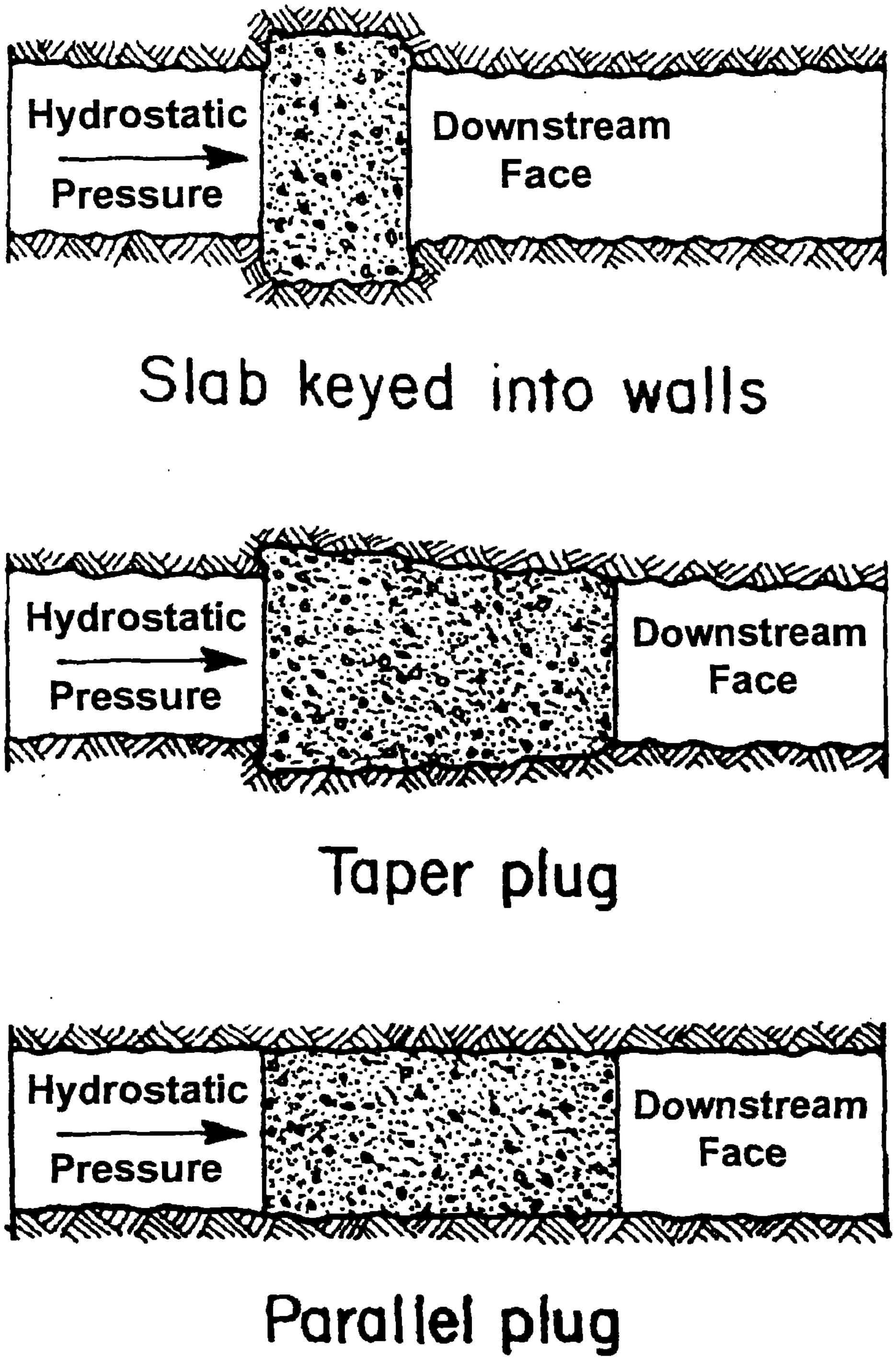
5) the corrosion rates for the piping and concrete and

6) the earthquake for the bulkhead area.

Of the 22 bulkheads that I have worked on, five did not successfully retain water. Three of those bulkheads were unsuccessful because of construction deficiencies and two because of unanticipated geologic conditions. One of the construction problems was repaired and one of the geologically deficient bulkheads redesigned and rebuilt to conform with the geology encountered.

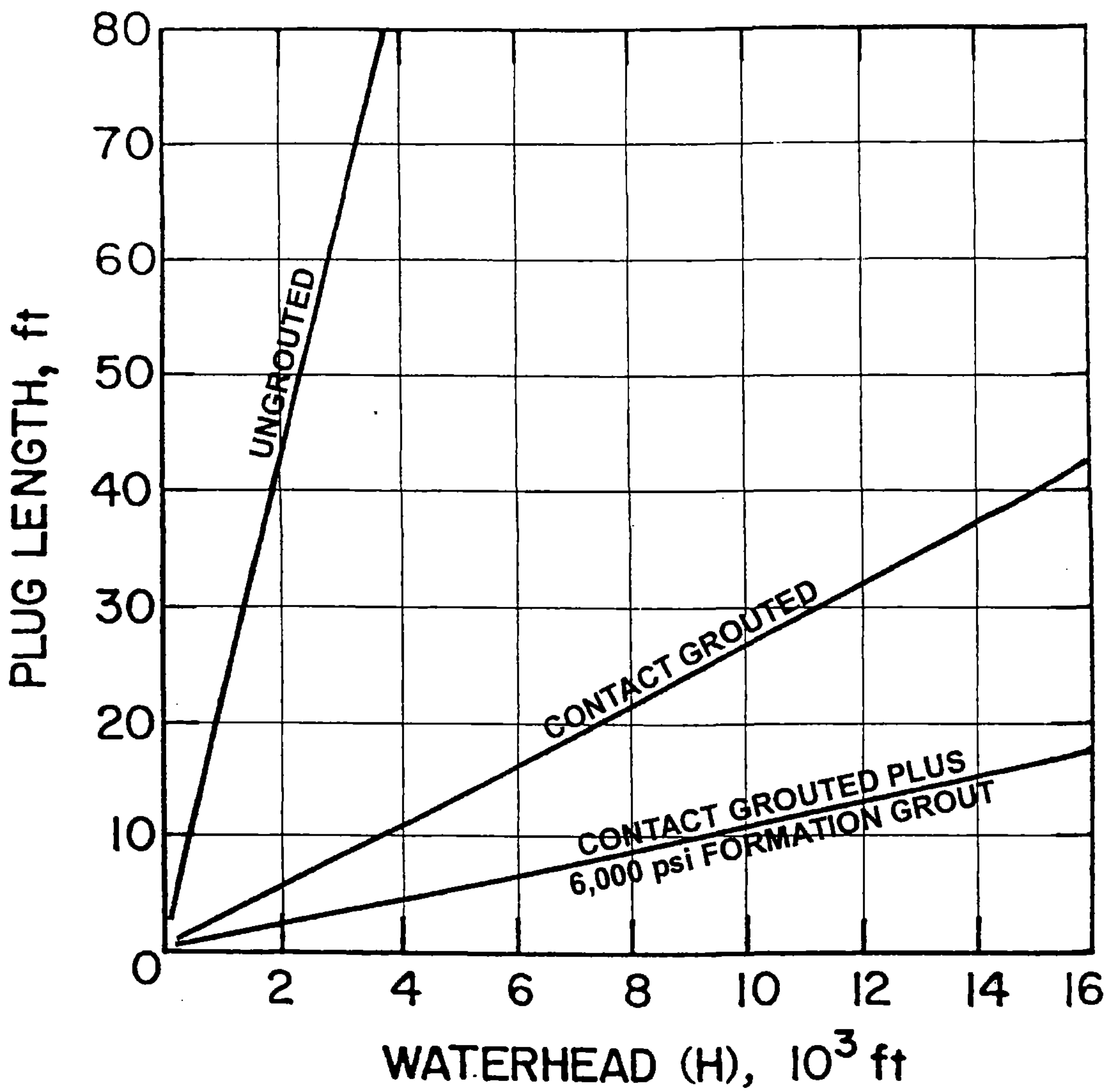
Low-pressure grouting of the concrete/rock contact is recommended to increase the hydraulic gradient resistance between the plug and the rock and to decrease the required length of the bulkhead. The shear strength of the concrete and the rock must exceed the perimeter shear stress developed by the maximum head. The bending stress at the downstream face must either be kept below allowable plain concrete design tensile strength or steel tensile reinforcement must he placed near the down stream face to support the potential tensile stresses. The bulkhead must be installed at a depth sufficient to prevent hydrofracing the formation and the loss of acid water to the formation joint system. Chemical attack of the bulkhead concrete must be resisted and the corrosion rate of the piping through the bulkhead must be balanced by sufficiently thick pipe walls to provide for the required minimum bulkhead life. The site specific earthquake loading hazard should be evaluated.

Figure 1. Types of bulkheads in use.



(Adapted from Garrett & Campbell Pitt, 1958)

Figure 2. Test results from experimental bulkhead (Garrett & Campbell Pitt, 1958).



Adapted from Garrett and Campbell Pitt's 1958 test results for experimental bulkhead, Factors of Safety of 1.00

Figure 3. Compressive strength results, Chandler Tunnel.

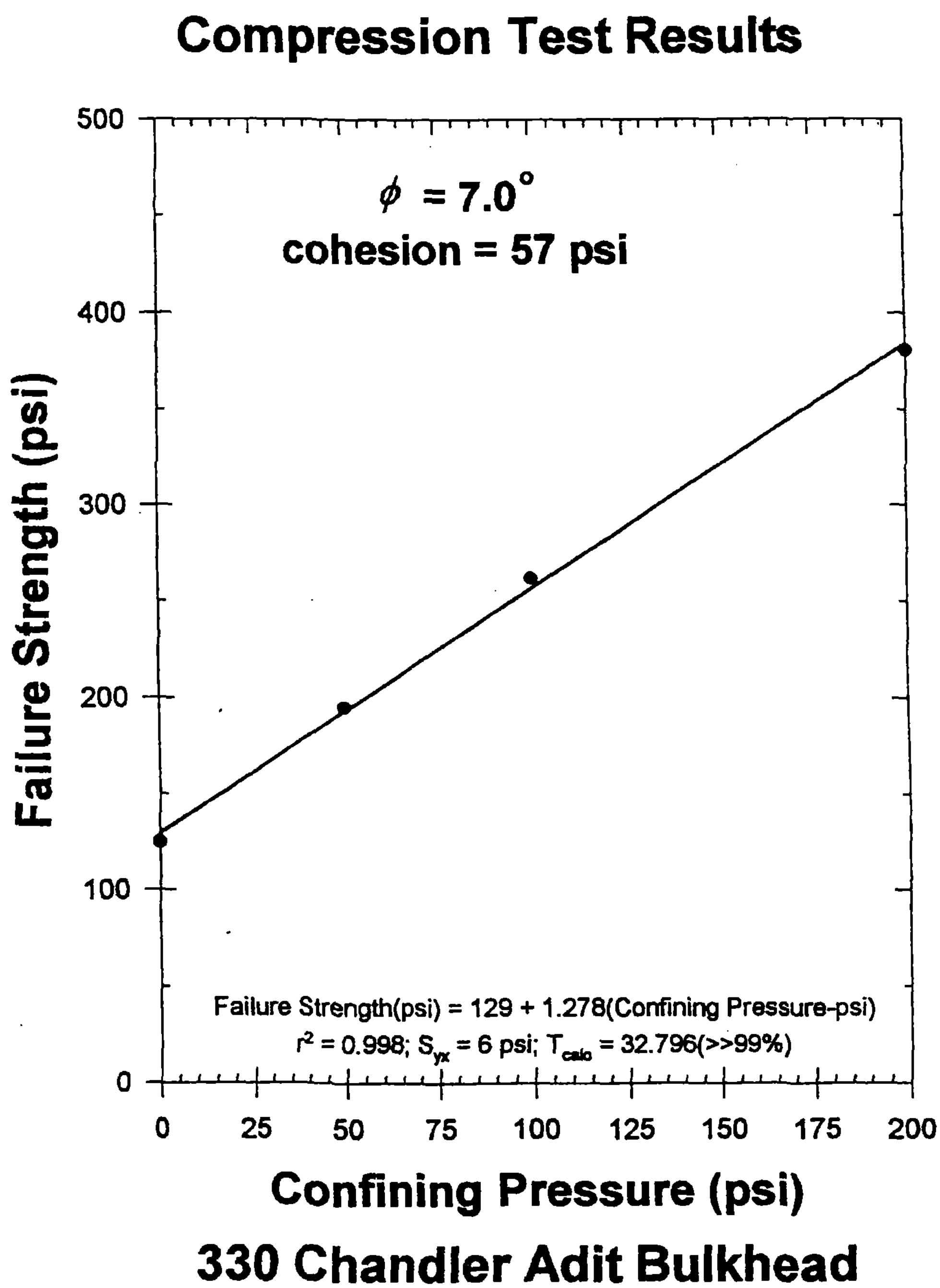


Table 1. Concrete selection based on sulfate concentration (ACI 318-95, Section 4.3.1)

TABLE 4.3.1—REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate exposure	Water soluble sulfate (SO_4) in soil, percent by weight	Sulfate (SO_4) in water, ppm	Cement type	Maximum water-cementitious materials ratio, by weight, normal weight aggregate concrete*	Minimum f_c', normal weight and lightweight aggregate concrete, psi*
Negligible	0.00-0.10	0-150	—	—	—
Moderate†	0.10-0.20	150-1500	II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	4000
Severe	0.20-2.00	1500-10,000	V	0.45	4500
Very severe	Over 2.00	Over 10,000	V plus pozzolan‡	0.45	4500

* A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 4.2.2).

† Seawater.

‡ Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Figure 4. Longitudinal cross section of American Tunnel Bulkhead.

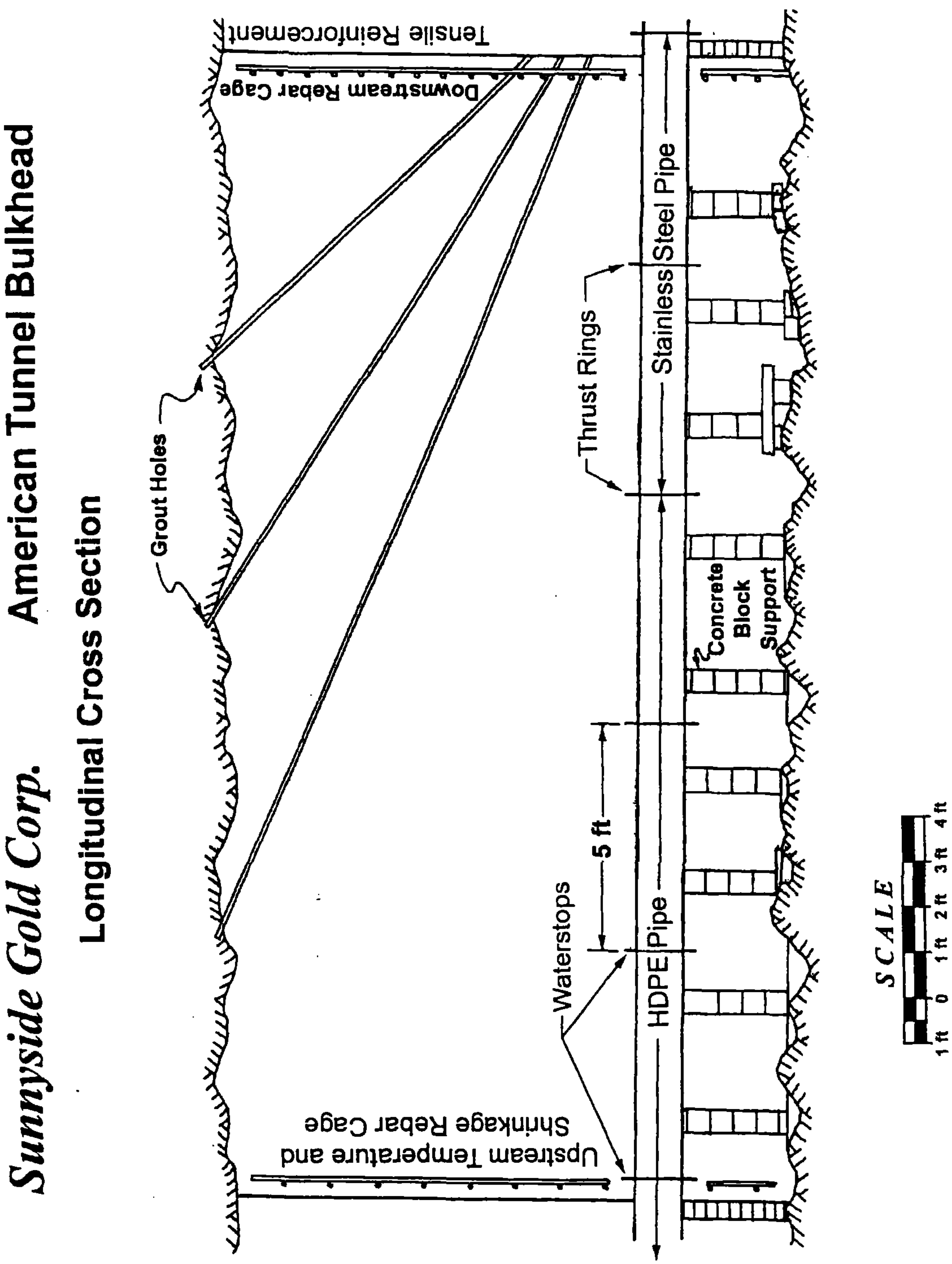
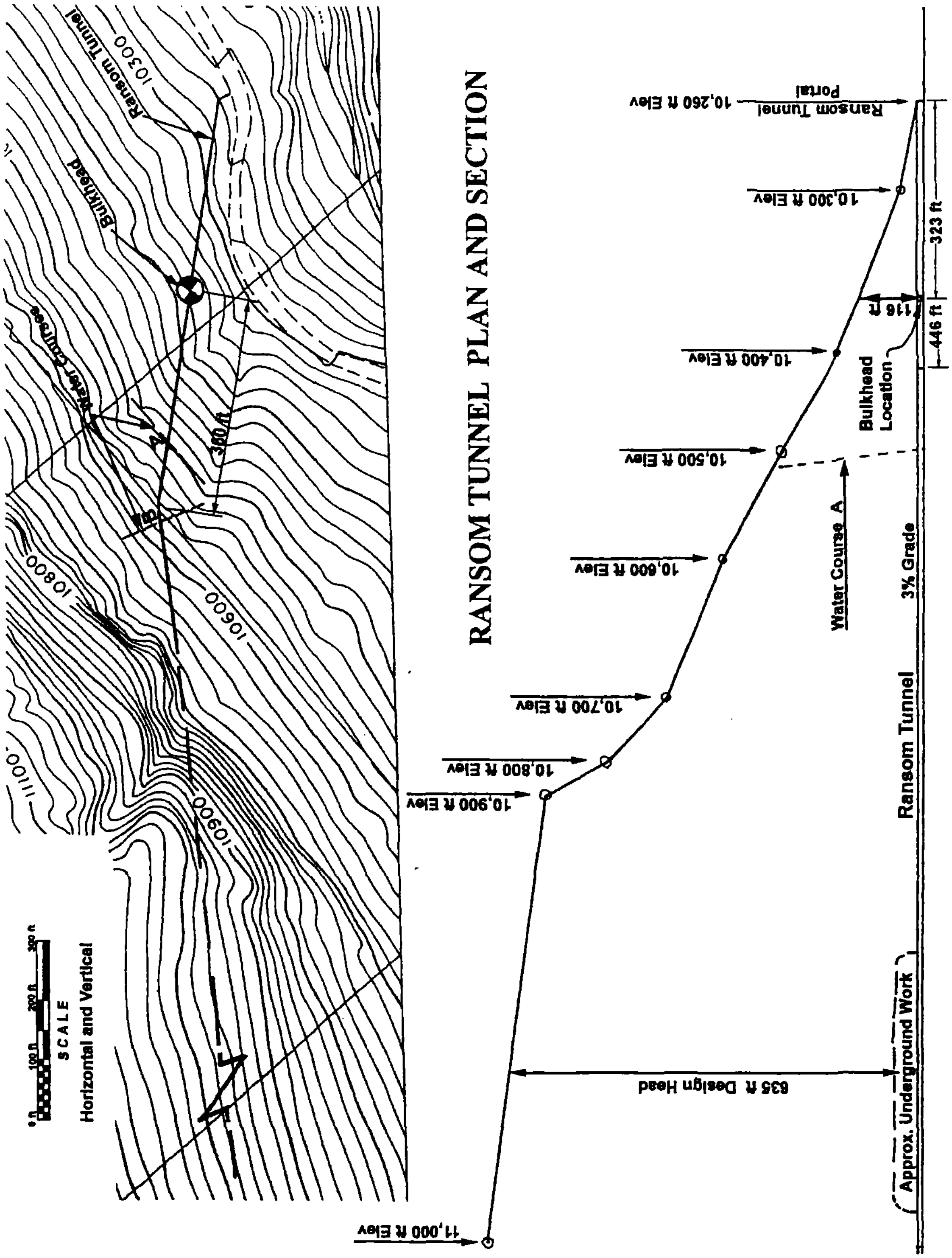


Figure 5. Ransom Tunnel Bulkhead location.



REFERENCES

Abel, J.F., Jr., 1993a, Bulkhead design for the Sunnyside Mine: consulting report to Sunnyside Gold Corp., 54 p

Abel, J.F., Jr., 1993b, Bulkhead design philosophy: consulting report to Intermountain Mine Services, Inc., 12 p

Abel, J.F., Jr., 1997, Bulkhead design for the Ransom Tunnel: consulting report to Sunnyside Gold Corp., 3 p

American Concrete Institute, 1977, Recommended practice for evaluation of strength test results of concrete (ACI 214-77): 23 p

American Concrete Institute, 1983, Recommended practice for evaluation of strength test results of concrete (ACI 214-77)(Reapproved 1983): 23 p

American Concrete Institute, 1989, Building code requirements for structural plain concrete (ACI 318.1-89) and commentary - ACI 318.1R-89: 14 p

American Concrete Institute, 1993, Guide for selecting proportions for high-strength concrete with Portland cement and fly ash (ACI 211.4R-93): 13 p

American Concrete Institute, 1995, Building code requirements for structural concrete (ACI 318-95) and commentary - ACI 318R-95: 369 p

American Institute of Steel Construction, 1989, Manual of steel construction, allowable stress design: 9th ed, AISC, Inc., 1121 p

J.D. Bredehoeft, R.G. Wolff, W.S. Keys & E. Shuter, 1976, Hydraulic fracturing to determine the regional in situ stress field, Piceance Basin: Colorado: GSA Bulletin, v 87, p 250-258.

Brown, A., Consultants, Inc., 1992a, Preliminary design of plug for the Reynolds Adit, Summitville Mine, Colorado: Report 1288B/920825, Aug 19, 33 p

Brown, A., Consultants, Inc., 1992b, Adit plugging case histories: Report 1288B/921105, Nov 5, 17 p

Brown, A., Consultants, Inc., 1992c, Model evaluation of the effectiveness of plugging the Reynolds Adit: Report 1288B/920813/R1, Nov 19, 26 p

REFERENCES (Continued)

Canadian Mining Jour, 1985, Anatomy of a crisis: Dec, p 10-11

Chekan, G.J., 1985, Design of bulkheads for controlling water in underground mines: USBM Info Circ 9020, 36 p

Coogan, J. & F.C. Kintzer, 1987, Tunnel plug design at Tyee Lake: Bulletin Assoc. Engineering Geologists, v 24, no 1, p 27-42

Einarson, D.S. & J.F. Abel, Jr., 1990, Tunnel Bulkheads for Acid Mine Drainage: Proc Symp on Unique Underground Structures, v 2, p. 71-1 to 71-20

Garrett, W.S. & L.T. Campbell Pitt, 1958, Tests on an experimental underground bulkhead for high pressures: Jour S. African Inst Mining and Metallurgy Congress, Oct, p 123-143

Garrett, W.S. & L.T. Campbell Pitt, 1961, Design and construction of underground bulkheads and water barriers: in Proc 7th Commonwealth Mining and Metallurgy Congress, v 3, p 1283-1301

Lancaster, F.H., 1964, Research into underground plugs Transvall and Orange Free State Chamber of Mines: Research Report 27/64, Aug, 130 p (Govt Mining Engr of S. Africa)

Lindeburg, M.R., 1989, Civil engineering reference manual: fifth ed, Professional Publications, Inc., 646 p

Loofbourow, R.L., 1973, Groundwater and groundwater control: in SME Mining Engineering Handbook, Section 26.7.4, Underground bulkheads and plugs, p 2646-2648

Louw, A., 1970, Ordeal by water: Mining Congress Jour, Part 1, Mar, p 43-56, Part 2, Apr, p 99-103

Merritt, F.S., 1983, Standard handbook for civil engineers: McGraw-Hill, 1578 p

Neukirchner, R.J. & D.R. Hinrichs, 1998, Effect of ore body innundation - a case study: Technical report provided by Eagle Engineering Services, Inc., 12 p

Obert, L. & W.I. Duvall, 1950, Generation and propagation of strain waves in rock: Part I, USBM Rpt of Investigation 4683

Petykopf, B.T., T.C. Atchison & W.I. Duvall, 1961, Photographic observation of quarry blasting: USBM Rpt of Investigation 5849

REFERENCES (Continued)

Singh, M.M., 1992, Mine subsidence: in SME Mining Engineering Handbook, ed 2, Chapt 10.6, Section 10.6.4.5 Hydrologic effects & Section 10.6.4.6 Nonmining damage, p 961-964

Troxell, G.E., H.E. Davis & J.W. Kelly, 1968, Composition and properties of concrete: 2nd ed, McGraw-Hill, 529 p

U.S. Bureau of Mines, 1992, Passive mine drainage treatment systems: Technology News, No 407A, AML #12A (reissue), 4 p

World Mining, 1969, How West Driefontein gold mine fought and won the flood battle: US Edition, Mar, v 5, no 3, p 18-23 & 36

Appendix A. Ransom Tunnel Bulkhead design calculations

Notation:

a = compression zone depth(in) minimum to balance rebar tension
A_s = area of rebar
b_w = web width (12 in)
C = comp bending force (lb)
D = dead load $\left(\frac{lb}{ft}\right)$
E = earthquake load $\left(\frac{lb}{ft}\right)$
E_m = earthquake mass $\left(\frac{lb\text{-}sec^2}{ft}\right)$
F = fluid load $\left(\frac{lb}{ft}\right)$
f'_c = concrete comp strength (3,000 psi)
f_{ct} = concrete tensile strength $\left(5\phi\sqrt{f'_c}\ \text{psi}\right)$
fy = rebar yield strength (60,000 psi)
H = design water head (635 ft)
I = moment of inertia
L = beam length or depth (10 ft)
M = bending moment (ft lb)
M_u = factored beam moment (ft lb)
m_c = minimum cover, form face to rebar surface (3.5 in)
T = tensile bending force (lb)
U_e = earthquake required strength
V_n = nominal shear force (lb)
V_u = factored shear force (lb)
W = bulkhead load (lb)
a = earthquake acceleration $\left(0.087\frac{ft}{sec^2}\right)$
ρ_g = pressure gradient $\left(\frac{psi}{ft}\right)$
ϕ = strength reduction factors
0.90 flexure rebar tension
0.85 concrete shear
0.65 plain concrete flexure
ω = uniform bulkhead load $\left(39,600\ \frac{lb}{ft}\right)$

b = beam width (1 in)
Bp = formation breakdown pressure (psi)
c = centroidal distance (in)
d = distance, extreme compression fiber to rebar centroid (in)
ΣE = total earthquake load (lb)
FS = factor of safety
$\sqrt{f'_c}$ = square root of f'_c
f'_s = concrete shear strength (psi)
g = acceleration due to gravity $\left(32.2\ \frac{ft}{sec^2}\right)$
h = tunnel height (10 ft)
$K = \left(3.5 - 2.5\frac{M_u}{d}\right)$
= tunnel width (10 ft)
M_n = nominal beam moment (ft lb)
M_{ue} = earthquake beam moment (ft lb)
S = section modulus (in^3)
S_l = line-of-sight distance (360 ft)
U = required strength $\left(\frac{lb}{ft}\right)$
V_c = concrete shear strength (lb)
V_s = rebar shear strength (lb)
v_s = rebar shear stress (psi)
ω = uniform load $\left(\frac{lb}{ft}\right)$
ρ = pressure head (275 psi)
$\rho_w = \frac{A_s}{b_w d}$
γ_w = water density (62.4PCF)
γ_c = concrete density (151PCF)
γ_r = rock density (173 PCF)
σ_s = flexure stress (psi)
Z = bulkhead design depth (ft)

Load factors (ACI 318, Sec 9.2.2, 9.2.3, 9.2.5)

Static fluid load factor (F) = 1.4;
Factor for fluid load under earthquake acceleration (F) = 1.05;
Earthquake accelerated load factor (E) = 1.40

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

Hydraulic pressure gradient:

Low pressure grouting of concrete-rock contact but not rock, gradient allowable = 41 psi/ft (Garrett & Campbell-Pitt, 1958, Chekan, 1985, p11), with factor of safety of 4

Ransom tunnel bulkhead, maximum pressure head

$$\rho = \frac{H\gamma_w}{144} = \frac{635(62.4)}{144} = 275 \text{ psi}$$

Required bulkhead length with low pressure grouting on concrete/rock bulkhead contact:

$$\mathbf{L} = \frac{\rho}{40} = \frac{275}{40} = \mathbf{6.9\ ft}$$

Pressure gradient with L = 8 ft $\quad \rho_g = \frac{\rho}{8} = \frac{275}{8} = 34.4$ psi/ft

Factor of Safety against water leakage along concrete/rock contact around 8-ft thick bulkhead is:

$$\underline{\mathbf{FS}} = \frac{41}{34.4} = \underline{\mathbf{1.19}}$$

Concrete shear on Ransom tunnel perimeter:

$$f_s' = 2\sqrt{f_c'} = 2\sqrt{3000} = 110 \text{ psi} \qquad \text{(ACI 318-95, Sec 11.3.1.1)}$$

$$L = \frac{\rho h l}{2(b+H)f_s'} = \frac{275(10)10}{2(10+10)110} = \frac{27500}{4400} = \mathbf{6.25\ ft}$$

$$W = \rho h = 275(10)10(144) = 3{,}960{,}000 \text{ lb}$$

$$v_s = \frac{W}{[2(b+H)]L(144)} = \frac{3960000}{[2(10+10)]8(144)} = 85.9 \text{ psi}$$

$$\underline{\mathbf{FS}} = \frac{f_s'}{v_s} = \frac{110}{85.9} = \underline{\mathbf{1.28}}$$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

Plain concrete deep beam bending stress design, Ransom tunnel (ACI 318-95, Sec 9.9.2.5, 18.4.1(b), & ACI 318-71, Sec 9.2.1.5)

Ransom Tunnel bulkhead, for 635-ft hydraulic head (275 psi pressure head):

$$\omega = U = 1.4\rho(144) = 1.4(275)144 = 55{,}400 \left(\tfrac{lb}{ft}\right)$$

$$M_n = \frac{\omega\ell^2}{8} = \frac{55400(10^2)}{8} = 692{,}500 \text{ ft·lb}$$

$$M_u = \frac{M}{.\ 5} \quad \frac{925}{.\ 5} = 1{,}065{,}000 \text{ ft·lb}$$

$$S = \frac{I}{c} = \frac{\frac{bL^3}{12}}{\frac{L}{2}} = \frac{\frac{1(L^3)(12^3)}{12}}{\frac{L(12)}{2}} = \frac{144L^2}{6}$$

$$f'_{cl} = 3\sqrt{f'_c} = 3\sqrt{3000} = 164 \text{ psi}$$

$$f'_{cl} = 164 = \sigma = \frac{M_u c}{I} = \frac{M_u}{S} = \frac{1065000}{\frac{144L^2}{6}} = \frac{44400}{L^2}$$

$L = \sqrt{\frac{44400}{164}} = \sqrt{271} =$ 16.5 ft, length required for plain concrete bulkhead.

$$\sigma_s = \frac{M_u}{S} = \frac{M_u}{\frac{144L^2}{6}} = \frac{1065000}{\frac{144(8^2)}{6}} = \frac{1065000}{1536} = 693 \text{ psi}$$

$$\mathbf{FS} = \frac{f'_{cl}}{\sigma_s} = \frac{164}{693} = \mathbf{0.24}$$

Therefore, 8-ft long plug must be reinforced.

Reinforced concrete deep-beam bending stress design, Ransom tunnel (ACI 318-95, Sec 9.3.2.3, Sec 9.3.2.3.: Wang & Salmon, 1985: Einarson & Abel, 1990)

$$C = \phi f'_c b_w = 0.85(3000)12a = 30600a$$

$$T = A_s f_y = 60000A_s$$

$$C = T; \qquad 30600a = 60000A_S; \qquad a = \frac{60000A_s}{30600} = 1.961A_s$$

$$M_n = \frac{\omega\ell^2}{8} = \frac{55400(10^2)}{8} = 692{,}500 \text{ ft·lb}$$

$$M_u = \frac{M_n}{0.9} = \frac{692500}{0.9} = 769{,}400 \text{ ft·lb } (9{,}233{,}000 \text{ in·lb})$$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

$M_u = A_s f_y(d - \frac{a}{2})$; $\quad d = L - m_c = 8(12) - 3.5 = 92.5$ in

$M_u = 60000A_s(d - \frac{a}{2}) = 60000A_s(92.5 - \frac{1.961A_s}{2}) = 5550000A_s - 58800A_s^2$

Therefore: $9{,}233{,}000 = 5{,}550{,}000A_s - 58{,}000A_s^2$

$58{,}800A_s^2 - 5{,}550{,}000A_s + 9{,}233{,}000 = 0$

As = 1.69 $\frac{in^2}{ft}$ steel area required

#10 bars (1.270 in^2 per bar) on 8-in c-c provides 1.905 $\frac{in^2}{ft}$ steel area

Check for adequacy

Allowable $M_u = -58{,}800A_s^2 + 5{,}550{,}0000A_s = 10{,}360{,}000$ in·lb

Design Mu = 9,233,000 in·lb

<u>**FS**</u> $= \frac{10360000}{9233000} = \underline{\mathbf{1.12}}$

Critical section shear strength for Ransom tunnel, 8-ft deep beam bulkhead

Deep beam defined as $\frac{\ell}{d} < 5$ (ACI 318-95, Sec 11.8.1). Critical section shear at 0.15l (1.28 ft) from ribside (ACI 318-95, Sec 11.8.5), with #10 bars on 8-in c-c, there will be 1.905 $\frac{in^2}{ft}$ of steel per ft of beam width, d = 92.5 in (7.71 ft).

Detailed shear strength at critical section (ACI 318-95, Sec 11.8.7)

$\frac{\ell}{d} = \frac{10(12)}{[8(12)-3.5]} = \frac{120}{92.5} = 1.30 < 5$ Therefore, reinforced concrete bulkhead is a deep beam for design!

v_n - nominal shear stress shall not be greater than $8\sqrt{f_c'}$ when $\frac{\ell}{d} < 2$
(ACI 318-95, Sec 11.8.4)

Limiting value: $v_n \le 8\sqrt{3000} \le 438$ psi $\quad V_n \le (v_n)b_w d \le (438)12(92.5) \le 486{,}200$ lb

$V_n = \frac{\omega\ell}{2} - (\frac{\omega\ell}{2})(\frac{0.15\ell}{0.5\ell}) = 0.35\omega = 0.35(55400)10 = 193{,}900$ lb

$V_u = \frac{V_n}{0.85} = \frac{193900}{0.85} = 228{,}100$ lb

$M_n = (\frac{\ell}{2})(0.15\,\ell) - \omega(0.15\,\ell)\frac{0.15\,\ell}{2} = 0.06375\frac{\omega\ell^2}{2} = 0.06375\frac{[55400(10^2)]}{2}$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

$M_n = 176{,}100$ ft·lb

$M_u = \frac{M_n}{0.9} = \frac{176100}{0.9} = 196{,}200$ ft·lb

$V_c = K\left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d$

$K = 3.5 - 2.5\frac{M_u}{V_u d} = 3.5 - 2.5\left[\frac{196300}{228000\left(\frac{92.5}{12}\right)}\right] = 3.5 - 0.28 = 3.22$

K cannot exceed 2.5 Therefore K = 2.5

$\rho_w = \frac{A_s}{b_w d} = \frac{1.905}{(12)92.5} = 0.001716$ Trial, #10 bars on 8-in centers, two-way

$V_c = K\left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d$

$V_c = 2.5\left[1.9\sqrt{3000} + 2500(0.001716)\frac{228100\left(\frac{92.5}{12}\right)}{196200}\right]12(92.5)$

$V_c = 2.5[104.1 + 38.45]1110 = 2.5[142.55]1110 = 395{,}500$ lb

Allowable $Vc \le \left(6\sqrt{f'_c}\right) b_w d \le (6\sqrt{3000})12(92.5) = 364{,}800$ lb (ACI 318-95, Sec 11.8.7)

Therefore, **FS** $= \frac{V_c}{V_u} = \frac{364800}{228100} = \mathbf{1.60}$

Ransom Tunnel bulkhead depth below surface (Z) required to prevent hydrofrac of rock around tunnel by 635-ft hydraulic head (275 psi pressure head): (Einarson & Abel, 1990)

$B_p = 3\sigma_{min} - \sigma_{max} = 2\sigma_{ovb} = 2Z\left(\frac{\gamma_r}{144}\right) = 2Z\left(\frac{173}{144}\right) = 2.403Z \text{ psi} = 275 \text{ psi}$

$\mathbf{Z} = \frac{275}{2.403} = \mathbf{114\ ft}$

Therefore, the bulkhead must be centered at least 320 ft inside the portal to develop 114 ft of overburden. Recommended bulkhead location from 319 ft to 327 ft inside the portal, for average distance from the portal of 323 ft and an average depth of 116 ft.

Earthquake bulkhead design; Load factors (ACI 318-95, Sec 9.2.2, 9.2.3, 9.2.5) Factor for fluid load under earthquake acceleration (F) = 1.05; Load factor for earthquake accelerated mass (E) = 1.40. Maximum credible earthquake acceleration (a) is 0.087 $\frac{ft}{sec^2}$.

$U = 1.05F + 1.40E$

Appendix A. Ransom Tunnel Bulkhead design calculations (Continued)

Mass (E) accelerated by maximum credible earthquake

$$E_m = \frac{S_l \gamma_w h\ell + Lh\ell\gamma_c}{g} = \frac{[360(62.4)10(10)+8(10)10(151)]}{32.2} = \frac{[2,246,400+120,800]}{32.2} = 73,520 \ \frac{\text{lb-sec}^2}{\text{ft}}$$

$$\Sigma E_m = E_m a = 73,520(0.087) = 6396 \text{ lb}$$

$$E = \frac{\Sigma E_m}{h} = \frac{6396}{10} = 640 \frac{\text{lb}}{\text{ft}}$$

Total load under earthquake acceleration

Ransom Tunnel bulkhead, for 635-ft hydraulic head:

$$\rho = \frac{H\gamma_w}{144} = \frac{635(62.4)}{144} = 275 \text{ psi}$$

$$F = \rho b_w(12) = 275(12)12 = 39,600 \ \frac{\text{lb}}{\text{ft}}$$

$$U_a = 1.05F + 1.40E = 1.05(39600) + 1.40(640) = 41,580 + 896$$

$$U_a = 42,480 \ \frac{\text{lb}}{\text{ft}}$$

Earthquake nominal beam bending moment

$$M_{na} = \frac{U_a \ell^2}{8} = \frac{42480(10^2)}{8} = 531,000 \text{ ft·lb}$$

$$M_{ua} = \frac{M_n}{0.9} = \frac{531000}{0.9} = 590,000 \text{ ft·lb } (7,080,000 \text{ in·lb})$$

Steel area required for earthquake loading:

$$58800A_s^2 - 5,550,000A_s + 7,080,000 = 0$$

$A_s = 1.29 \ \frac{\text{in}^2}{\text{ft}}$ Steel area required to resist maximum credible earthquake loading.

#10 bars on 8-in c-c provide 1.905 $\frac{\text{in}^2}{\text{ft}}$ steel area

Check for adequacy

Allowable $M_{ua} = -58,800A_s^{\prime} + 5,550,000A_s = 10,360,000 \text{ in} \cdot \text{lb}$

Design $M_{ua} = 7,080,000 \text{in} \cdot \text{lb}$

<u>FS</u> $= \frac{10360000}{7080000} = \underline{1.46}$ against earthquake loading.

Appendix B. Some acid mine drainage bulkheads installed in Colorado

Mine/Location	Distance from Portal (ft)	Depth below Surface (ft)	Design Head (ft)	Years of Service	Comments
Eagle Mine, Gilman					Numerous seeps (9), 7 along Rock Creek, equilibrium water level 80 ft lower than design, poor quality initial water seeps, improved over time
Adit 6, '86	80	≈ 70	246	14	
Adit 5, '86	200	≈125	172	14	
Adit 7, '87	150	≈100	87	13	
Newhouse, '87	≈150	≈ 90	112	13	
Ben Butler Adit, '90	≈200	≈ 60	110	9	
Tip top Adit, '90	≈100	≈ 50	118	9	
Star of the West Incline, '90		≈130	101	9	Internal
Comet Claims, Placer Gulch, Silverton, '91					Initial <1.5 gpm leak Lower Level along fracture zone east side of plug, 1-in HDPE compression fitting on pressure gage line failed 2nd melt season
Lower Level	250	230	520	2	
Upper Level	150	122	295	2	
Thompson Creek Coal & Coke #1 Mine	30	20	Unk	< 8	Water spurting around thin (≈ 18-in), ungrouted plug
Sunnyside Gold Corp.					
American Tunnel	7950	2130	1550	4.5	1015-ft current head, ≈+30'/yr, pH 2.8 start,5.4 now, w/20 tons lime placed, 5gpm initial leakage reduced to drips by regrouting
Terry Tunnel	3800	1160	650	3	≈ 109-ft current head, no leaks

Appendix B (Continued). Some acid mine drainage bulkheads installed in Colorado

Mine/Location	Distance from Portal (ft)	Depth below Surface (ft)	Design Head (ft)	Years of Service	Comments
Summitville Mine					
Reynolds Adit	1250	425	350	7	Minor dripping at downstream face, high strength alloy bolts severely corroded in 2 yrs, leakage through fracture system starting ≈ 100 ft downstream from bulkhead in 3120 psi rock
Chandler Adit	330	95	175	≈ 4	Initial 7-ft bulkhead failed at ≈ 85 ft head along 1-ft wide roof fault, overall 129 psi rock 20-ft extension, no leakage over ≈ 4 yrs

Appendix B (Continued). Some water impoundment bulkheads installed outside Colorado

Mine/Location	Distance from Portal (ft)	Depth below Surface (ft)	Design Head (ft)	Years of Service	Comments
Walker Mine, Plumas Cty, CA*	2700	810	500	13	Low permeability rock, maximum head 210 ft equilibrium at 120 ft of head, minor leakage
Mammoth Mine, Shasta Cty, CA*					
Friday Louden Tunnel	613	150	670	8	No leakage, ≶ 350-ft max head
Lower Gossan	200	100	300	7	No plug leakage, unknown head, pressure loss thru formation fractures
Upper Gossan	250	100	140	7	No leakage, unknown head
Keystone Mine, Shasta Cty, CA*					
Keystone 275	100	75	138	7	No leakage, unknown head
Keystone East Adit	400	250	288	7	No leakage, unknown head
Keystone 400 Level	200	100	450	0	No initial retention, 20-ft OD 130 psi grout ring added, failed to hold water. tight jointing (~2-in) weathered
Stowell Mine, Shasta Cty, CA*	200	200	300	7	Two portals w/plugs installed, No leakage, unknown head
Tyee Lake, AK Hydropower Tunnel	1500	790	1338	12	33gpm initial leakage, reduced to 11gpm by regrouting contact

* - Acid mine drainage bulkheads

Appendix B (Continued). Some historical worldwide records of bulkhead life

Mine	Length (ft)	Year Built	Design Head (ft)	Years of Service	Comments
CMR-6 Shaft 9 Level West, RSA	17	1953	830	45	Isolation bulkhead
East Daggerafontein 30 Haulage North and South, RSA	28	1949	1500	49	2 isolation bulkheads
Virginia 31 Haulage South, RSA	63	1957	3810	41	Isolation bulkhead, no leaks
Free State Geduld 47 Level, RSA	46	1955	1910	43	Emergency, 1st parallel sided bulkhead, full load in 72 hrs
Govt. G.M. Areas, RSA	5	1945	230	53	Isolation bulkhead
Sub Nigel, RSA	11	1953	459	45	Isolation bulkhead
West Dreifontein 10 & 12 Levels 4 bulkheads, RSA	60	1968	4000 (3740 actual)	30	Emergency 67,000 gpm inrush, pH 3.8, 2500 psi sand-concrete, alloy steel severely corroded 14-ft high, 12-ft wide
West Dreifontein	7.7	1958	15690	<1	Experimental bulkhead, 400 gpm leakage
Rocanville Mine, PCS Saskatchewan, CAN	87	1985	3000	13	Emergency 6,250 gpm inrush, potash mine from overlying aquifer, 8-ft high, 20-ft wide
Mammoth Mine, Shasta Cty, CA					
Friday Louden Tunnel	6	1980	212	≈1	Insufficient strength for 670-ft redesign head. Removed & rebuilt
Walker Mine, Plumas Cty, CA	15	1987	500	13	Maximum head 210-ft, minor leakage along contact and through formation

Detailed Work Plan and Benchmark Funding Schedule

#1 Send surface topography maps, tunnel long sections and any pertinent information to design engineer for guidance on probable plug location and plug size.

#2 Advance $15,000 per portal.
Open portal (portals if multiple levels) for 1 yd LHD access, establish ventilation and any other appropriate safety measures to secure portal and tunnel for selection of actual plug site and collecting rock samples. Build sediment traps as needed to control sludge that will be discharged.

#3 Close out #2 costs and advance $20,000.
Obtain Engineering design and submit to the Division of Minerals and Geology ("DMG") and the Water Quality Control Division ("Division") for approval.

#4 Establish coffer dam site, build coffer dam and divert water through piping.

#5 Close out #3 and #4 and advance $30,000.
Excavate plug area to solid rock, remove all loose rock and clean back, ribs and sill to remove mud, oxidation and other deposits to insure bonding of the concrete. Sand blasting works. Confirm size and taper assumptions used for design.

#6 Construct forms and place rebar. Arrange with DMG and the design engineer for pre-pour inspection. Determine grout pattern targets, mark hole collars to miss rebar and record drill angles and lengths to rock contact.

#7 Close out #5 and #6 and advance $10,000.
Place any alkaline material in the area between bulkhead and coffer dam planned for plug protection. Setup for pour and pour. Sample concrete for 7 day and 28 day tests during pour to confirm design strength has been met.

#8 Strip forms and drill holes for low pressure contact grouting. Grout holes until refusal.

#9 Close valve. Grout valve and close portal if permanent closure is selected by owner. Submit construction certification report to DMG and the Division. Close out #7, #8 and #9 and distribute Remaining Funds in accordance with the terms of this Agreement.

DISTRICT COURT, CITY AND COUNTY OF DENVER, COLORADO Court Address: 1437 Bannock Street Denver, Colorado 80202	
Plaintiff: SUNNYSIDE GOLD CORPORATION, **Defendant:**COLORADO WATER QUALITY CONTROL DIVISION OF THE COLORADO DEPARTMENT OF PUBLIC HEALTH AND ENVIRONMENT	▲ COURT USE ONLY ▲ Case Numbers: 94 CV 5459 Div.: Ctrm.: 7

ORDER GRANTING JOINT PETITION FOR FOURTH AMENDMENT TO CONSENT DECREE

THIS COURT, having reviewed the Joint Petition for Fourth Amendment to Consent Decree, and thereby being advised in the premises, GRANTS the Joint Petition and ORDERS the Consent Decree to be modified as follows:

1. Appendix A to the Consent Decree is modified to be in accordance with the Appendix A submitted with the Joint Petition;

2. Paragraph 9.c. of the Consent Decree is modified to be consistent with the agreement to transfer ownership of the water treatment facility to the Gold King Mines Corporation ("Gold King"). Once the water treatment facility is transferred to Gold King and CDPS Permit No. CO-027529 is terminated or transferred to Gold King by the Colorado Water Quality Control Division ("Division"), Sunnyside Gold Corporation ("SGC") obligation to continue operation of the water treatment facility to treat Cement Creek or any seepage from the American Tunnel (and the reclamation of the ponds and surface disturbances) will terminate under the Consent Decree and, accordingly, paragraph 14.f of the Consent Decree will be deleted at that time.

3. Paragraph 10 of the Consent Decree is modified so as to require only the monitoring contained in Appendix A and any applicable DMG and CDPS permits;

4. In addition, SGC will fund or implement the following additional remediation projects:

a. Provide a total of $500,000, which the parties anticipate will be more than adequate, for plugging the Mogul and the Koehler Mines by Gold King or another entity to be approved by the Division. The sealing of the Mogul and Koehler Mines would be in accordance with the workplans attached to the Joint Petition as Appendix B, and following execution of agreements with the owners of those mines and Gold King allowing and providing the terms for the plugging;

b. Provide $172,000 to Gold King for water quality improvement projects, including a liner at the Howardsville Cell No. 1 Mine Tailing, installation of a pipeline from the Gold King mine to the water treatment facility and water treatment at the American Tunnel treatment plant;

c. SGC will remove the power plant tailings; and

d. SGC will build a passive treatment wall at the southwest edge of Tailings Pond No. 4.

5. The Division shall notify the financial institution that has issued the letter of credit for financial surety referenced in paragraph 25 of the Consent Decree, that the letter of credit, $5,000,000 (Five Million Dollars) shall be released in full. The letter of credit funds shall be used for, but not be limited to, the funding of the projects referenced in paragraph 4 above;

DATED this ______ day of _____________, 2002.

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District Court Judge